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CONTENTS

April, 1959

Papers

	Page
Physico-Chemical Properties of Soils: Clay Mineral by R. E. Grim	1
Physico-Chemical Properties of Soils: Ion Exchange Phenomena by A. W. Taylor	19
Physico-Chemical Properties of Soils: Soil-Water Systems by I. Th. Rosenquist	31
Physico-Chemical Properties of Soil: Role of Soil Technology by T. William Lambe	55
Discussion	71
Discussion	103

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PHYSICO-CHEMICAL PROPERTIES OF SOILS: CLAY MINERALS

Ralph E. Grim[1]

FOREWORD

This symposium on the physico-chemical properties of soils was arranged under the auspices of the Committee on Mechanical Properties of Materials, of the Engineering Mechanics Division, sponsored jointly with the Soil Mechanics and Foundations Division.

Its objective was to summarize for civil engineers the current status of knowledge in this field and to stimulate further research and applications of this comparatively new and fundamental tool of soil technology.

The four papers in this symposium have been published as Proc. Papers 1998, 1999, 2000, and 2001.

ABSTRACT

Modern concepts of the structure, composition, and origin of clay minerals and current procedures for their identification are reviewed.

INTRODUCTION

Prior to about 1925, when the first X-ray diffraction studies were made of clay materials, there were no adequate analytical tools for investigating clays and soils. As a consequence there were many different concepts regarding the components of clays. Chemical analyses had shown that such materials were composed of aluminum, silicon and water frequently also with iron,

Note: Discussion open until September 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1998 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. SM 2, April, 1959.

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alkalies, and alkaline earths, but there was no precise knowledge of how these elemental components were combined in the clays and soils.

After the use of X-ray techniques, other procedures such as differential thermal analysis, electron microscopy, and infra-red absorption came into use for the study of clay materials. With adequate research tools available, many persons in many fields became interested in clay research, and a vast new literature came into being on the composition and atomic structure of the fundamental components of clay materials, on the controlling factors for the origin of such materials, and on the fundamental factors determining the properties of clays and soils. Thus for the first time, such properties as plasticity could be investigated with some precise knowledge of the composition and structure of the components of the materials in question.

These researches have established to the satisfaction of substantially all students of clay and soils that argillaceous materials are composed essentially of extremely small particles (of the order of a micron or less in diameter) of one or more members of a small group of minerals that have come to be known as the clay minerals. Chemically the clay minerals are silicates of aluminum and/or iron and magnesium. Some of them also contain alkalies and/or alkaline earths, as essential components. These minerals are dominately crystalline, i.e., the atom composing them are arranged in definite geometric patterns. Examples are known of argillaceous materials containing substantial amounts of amorphous material, but the evidence is overwhelming that amorphous materials are not significant components of most such materials, and amorphous material cannot be called upon as a general explanation of the properties of clays and soils.

Most of the clay minerals have sheet or layered structures. Some of the clay minerals have elongate tubular or fibrous structures. Argillaceous materials then can be considered as essentially made up of extremely small particles, each one of which is either a book of sheet-like units or a bundle of tubes or fibers. Individual soils or clays may contain more than one kind of book-like units or a mixture of books and bundles of tubes or fibers. Also as will be mentioned presently, some components other than the clay minerals are also usually present in varying amounts in clays and soils.

Structure and Composition of the Clay Minerals

There are two fundamental building blocks for the clay mineral structures. One is a silica tetrahedral unit, Fig. 1a, in which four oxygens or hydroxyls

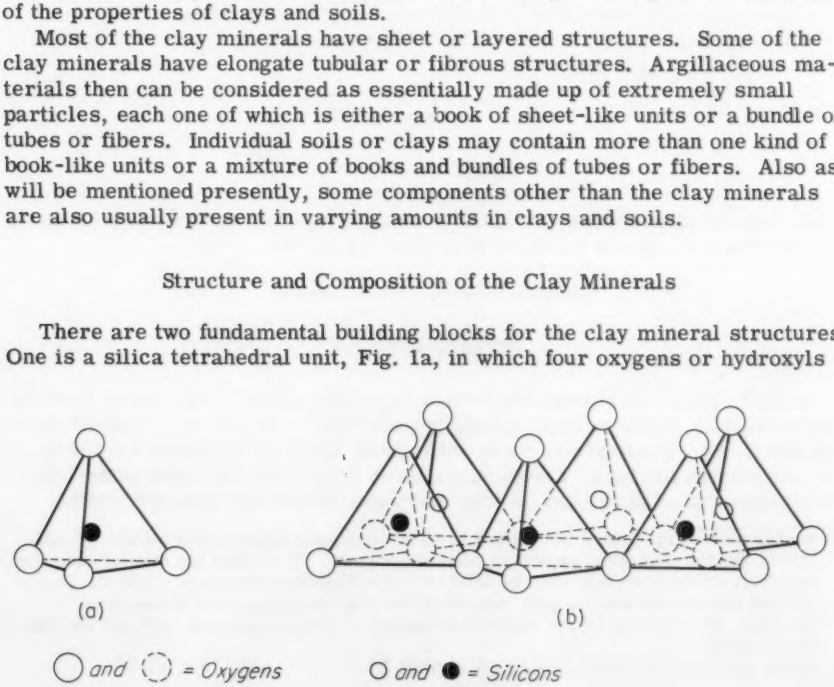

Fig. 1

having the configuration of a tetrahedron enclose a silicon atom. The tetrahedra are combined in a sheet structure so that the oxygens of the bases of all the tetrahedra are in a common plane, and each oxygen belongs to two tetrahedra, Fig. 1b. The silica tetrahedral sheet alone may be viewed as a layer of silicon atom between a layer of oxygens and a layer of hydroxyls (tips of the tetrahedra).

The second building block is an octahedral unit in which an aluminum, iron or magnesium atom is enclosed in six hydroxyls having the configuration of an octahedron, Fig. 2a. The octahedral units are put together into a sheet structure, Fig. 2b, which may be viewed as two layers of densely packed hydroxyls with the cation between the sheets in octahedral coordination.

Kaolinite

The kaolinite structural unit is an alumina octahedral layer with a parallel superimposed silica tetrahedral layer intergrown in such a way that the tips of the silica sheet and one of the layers of the octahedral unit form a common sheet, Fig. 3. The kaolin unit may be viewed as a succession of layers of oxygens, silicons, oxygens and hydroxyls, aluminums and hydroxyls. This unit is about 7A thick, and extends indefinitely in the other two directions, i.e., the flat dimension of the sheet. The kaolinite mineral is a stacking of such 7A thick sheets—the structure is like that of a book with each leaf of the book 7A thick. Successive 7A layers are held together with hydrogen bonds. The mineral cleaves fairly easily along the plane surface of the 7A units.

There is substantially no substitution of one cation for another within the kaolinite structure so that the mineral has the fixer formula $(OH)_8\ Al_4\ Si_4\ O_{10}$. It is of importance that in some clays, the kaolinite particles are composed of 7A units very regularly stacked one above the other, whereas in others the stacking is somewhat random.

Halloysite

Structurally halloysite is similar to kaolinite in being composed of sheet units about 7A thick, each sheet being made up of one silica tetrahedral layer and one alumina octahedral layer. Halloysite differs from kaolinite in that successive 7A units are more randomly stacked one above the other and a single molecular layer of water may enter between the 7A units. With a molecular layer of water between each sheet, the mineral has the composition $(OH)_8\ Al_4\ Si_4\ O_{10}\ 4H_2O$; without the layer of water the composition is the same as that of kaolinite. As the mineral occurs in nature, a molecular layer of water may not be fully developed between all 7A layers.

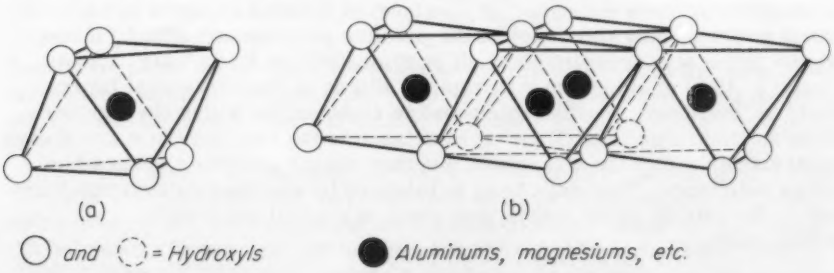

Fig. 2

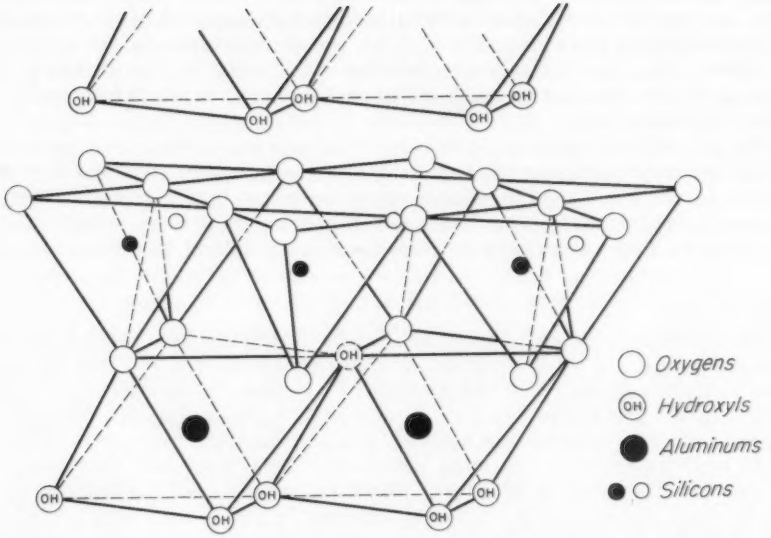

Fig. 3

A further structural factor of halloysite revealed by electron micrographs is that the mineral occurs in elongate units which in many cases appear to be tubes (Bates et. al. 1950) as if the sheet layers had rolled up, Fig. 4.

Montmorillonite

This mineral is made of sheet-like units, each unit being composed of two silica tetrahedral sheets and one octahedral sheet. The octahedral sheet is between the silica sheets with the tips of the tetrahedra pointing toward the octahedral sheet. The tips of each tetrahedral sheet and an hydroxyl layer of the octahedral sheet are intergrown to form a single layer. The montmorillonite structure can be viewed as a sheet structure with each sheet composed of successive layers of oxygens, silicons, oxygens and hydroxyls, aluminums (or irons or magnesiums), oxygens and hydroxyls, silicons, and oxygen, Fig. 5. The thickness of the sheet is about 9.5A and the dimensions in the other two directions are indefinite.

The octahedral elements may be aluminum, iron, or magnesium, or a combination of these elements. If aluminum is the sole occupant of the octahedral positions only two thirds of the possible positions are filled. If magnesium is the sole occupant about all positions will be filled. Also a small amount ($< 15\%$) of aluminum may replace silicon in the tetrahedral layers. There is, therefore, considerable possible replacement within the structure of the montmorillonites. Of great importance is the fact that there are always replacements within the structures, and they always produce a net positive charge deficiency. This deficiency is balanced by adsorbed cations which are held on the outside of the sheets, and which in general are readily exchangeable.

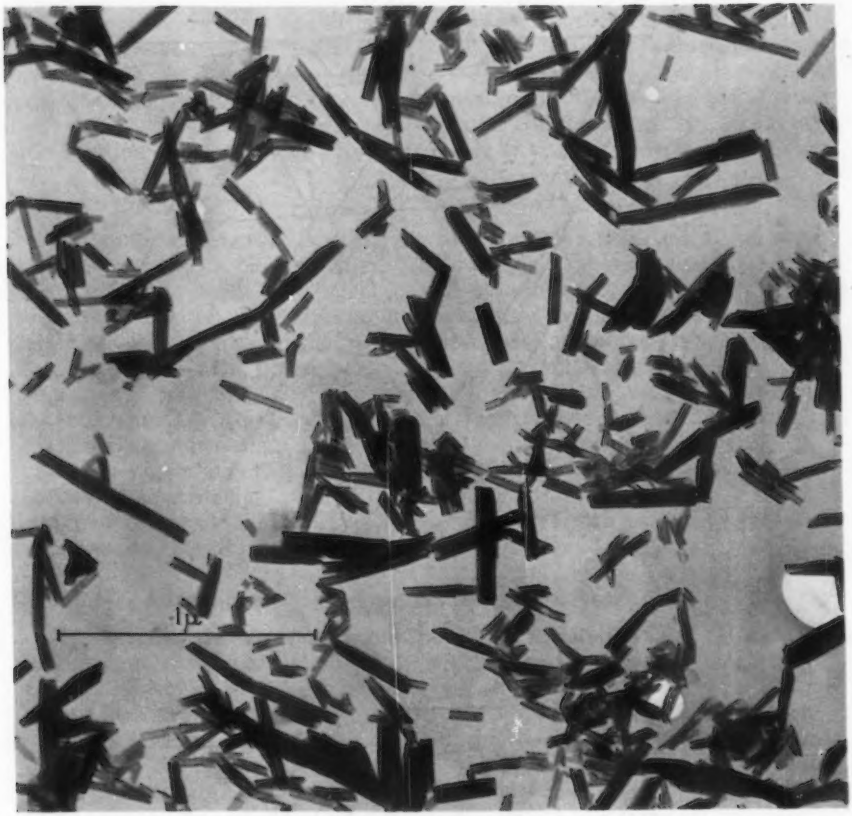

Fig. 4

It is difficult to write a general chemical formula for the montmorillonite minerals, but the following will serve as examples of common compositions (Ross and Hendricks, 1945):

$$(OH)_4\ Si_8\ (Al_{3.34}\ Mg_{66})\ O_{20}$$
$$(OH)_4\ (Si_{7.34}\ Al_{66})\ Fe_4\ O_{20}$$
$$(OH)_4\ (Si_{7.34}\ Al_{66})\ Mg_6\ O_{20}$$

Specific mineral names have been applied to varieties of montmorillonite with distinctive chemical compositions. Thus saponite is the name when magnesium is about the sole octahedral cation; nontronite when iron fills these positions.

The 9.5A sheets are stacked one above the other like the leaves of a book. There is very little bonding force between successive sheets and water may enter between the sheets causing the mineral to swell. About 80 per cent of the balancing cations also are between the sheets. The water layer may be of substantially any thickness depending somewhat on the nature of the adsorbed

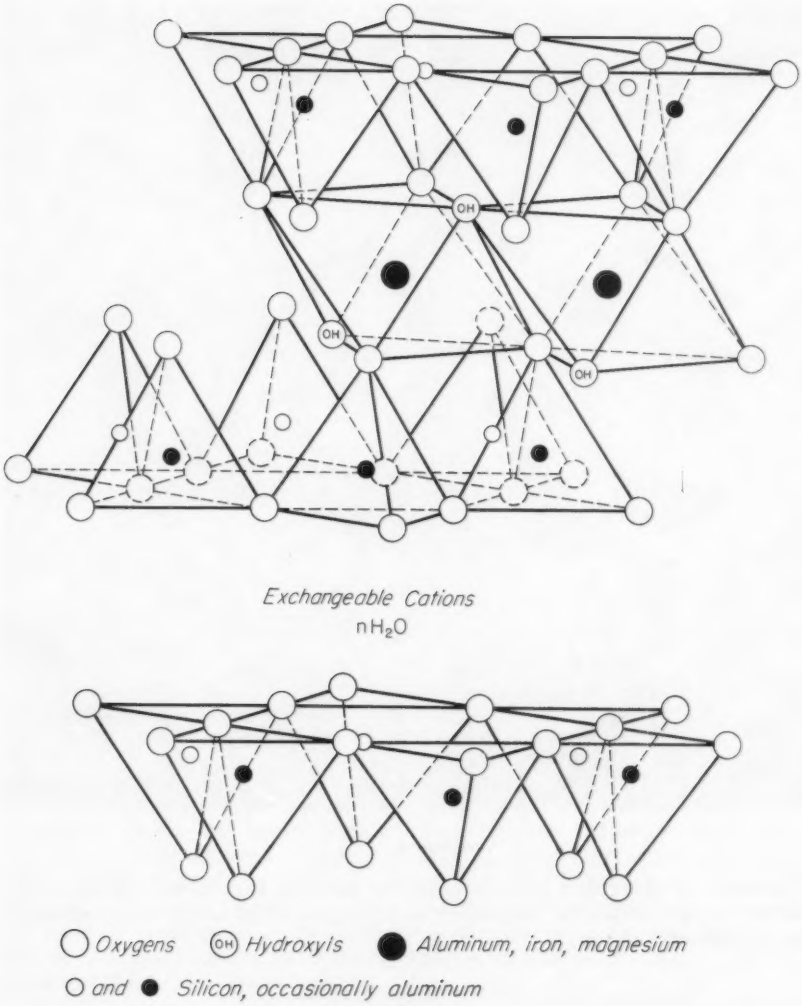

Fig. 5

cation and on the water available. In the presence of an abundance of water, the mineral can in some cases about split up into individual unit layers 9.5A thick.

Illite

The structure of illite is similar to that of montmorillonite except that there is always substantial (20%+) replacement of silicons by aluminums in the tetrahedral layers and potassiums are between the layers serving to balance the charges resulting from the replacement and to tie the sheet units together. The structure does not swell by the introduction of water between

successive sheets as does montmorillonite. The population of the octahedral positions can vary from one illite to another as can the amount of aluminum replacing silicon and the balancing potassiums. Illite is similar to the micas in structure and composition, but differ from them mainly in containing less potassium, slightly higher silicon to alumina molecular ratios, and in being less well ordered. A general formula (Grim, Bray, and Bradley, 1937) for the composition of illite is $(OH)_4\ K_y\ (Si_{8-y}\ Al_y)\ (Al_4\ .\ Mg_6\ .\ Fe_4\ .\ Fe_6)\ O_{20}$.

Chlorite

The chlorite structure is composed of a regular alternation of two structural units; one unit is like that of the micas having a central octahedral sheet enclosed in two silica tetrahedral sheets, the other unit is composed of magnesium (or iron or aluminum) atoms surrounded by six hydroxyls in octahedral coordination arranged together in a sheet structure. The latter unit (when Mg is the cation) has the structure of the mineral brucite. The chlorite structure then is a regular succession of mica, brucite, mica, brucite, etc., layers. The clay mineral chlorites are less well ordered and somewhat more variable in composition than the large museum specimens of the mineral. A general formula for chlorite is $(OH)_4\ (Si{\cdot}Al)_8\ (Mg{\cdot}\ Fe)_6\ O_{20}$ - $(Mg{\cdot}\ Al)_6\ (OH)_{12}$. The structural relations of kaolinite and chlorite are such that it is sometimes impossible to distinguish between them in a soil composed of a complex mixture of many clay minerals.

Sepiolite - Attapulgite

These minerals are composed of ribbon-like units (Bradley, 1940). Each ribbon is about 9.5A thick, 10A wide, and of indefinite length. A section across the 9.5A direction would show two silica tetrahedral layers with the tips of the tetrahedron pointing inward and enclosing an octahedral unit in which magnesium was the sole or at least principal occupant of the octahedral positions. The lath units are fastened together at their edges in such a way as to make an aggregate in which there are alternations of open channels and laths, Fig. 6. The general formula for attapulgite is $(OH_2)_4\ (OH)_2\ Mg_5\ (Si\ \ Al)_8\ O_{20}\ 4H_2O$. Sepiolite differs from attapulgite in having less aluminum replacing magnesium and in the width of the laths (Nagy and Bradley, 1955).

Allophane

This is the name applied to so-called amorphous components of clay minerals. It should not be considered that such minerals are composed of completely unorganized oxygens, silicons, etc., but rather that the tetrahedral and octahedral units are not arranged in very regular sheet structure, or the individual sheets are not arranged together with sufficient order or in sufficient size to produce diffraction effects. That is, a material can appear to be amorphous to X-ray or even electron diffraction and still have some considerable ordered arrangement of its elemental components. The absence of substantial order in the allophanes would mean of course that the composition is variable.

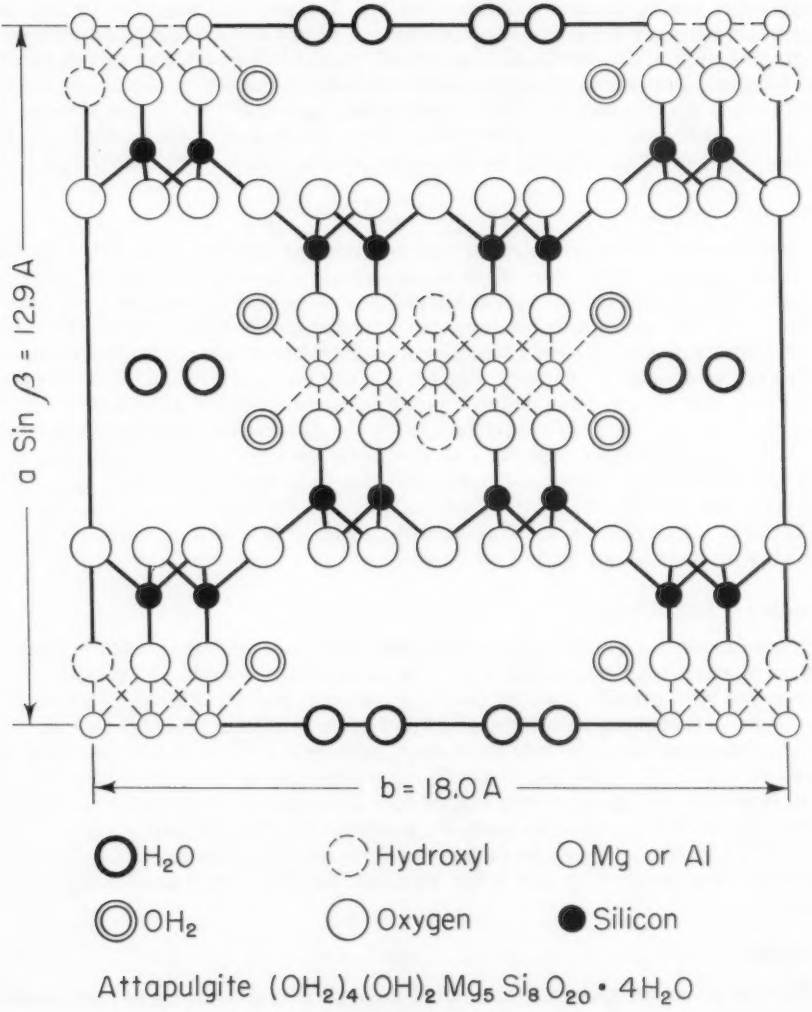

Fig. 6

Mixed Layers

Except for attapulgite-sepiolite, the clay minerals are sheet structures with many similarities. As a consequence they can occur in mixed layer structure in which a single or a few layers of one mineral e.g. illite, is interlayed with another mineral, e.g. montmorillonite. The mixed-layering can be a regular alternation of a fixed number of different unit layers or an alternation of layers of a random number of mixed layers. Such mixed-layer structures are now known to be very common particularly in soils and recent sediments. Their existence adds greatly to the difficulty of determining the

clay mineral composition of soils and clays, and to the complexity of naming and classifying the clay minerals.

It should be emphasized that the various clay minerals represent groups of minerals rather than individual species. Thus within the group of illite, for example, there are variations in composition, crystal order, etc. A result is that all illites do not have the same composition or properties. This is sometimes forgotten when a detailed study of the properties of one illite clay is made with the assumption that the results will apply to all illites. Such researches must include various illites in order to produce results of general applicability.

Non-Clay Mineral Components of Argillaceous Material

In addition to the clay minerals, argillaceous materials generally contain varying amounts of other minerals. The components may be called non-clay minerals because they are not essential components of clay materials, i.e., they are not essential for the development of such properties as plasticity that are fundamental to the nature of clays.

Quartz is the commonest non-clay mineral. Others frequently present are calcite, sulfides, feldspar and micas. In the case of the micas there is no absolute separation between fine grained clay mineral micas, like illite, and coarse grained muscovite or biotite which may not contribute substantially to the properties of the clay material. Frequently a particle size fractionation at about two microns will separate the clay mineral in the finer and the non-clay mineral in the coarser fraction. There are, however, instances in which quartz occurs in particles much less than two microns in diameter, and other cases in which kaolinite occurs in particles apparently coarser than two microns.

It should not be considered that the non-clay minerals are unimportant in influencing the properties of a clay or soil. Frequently the non-clay minerals serve only to dilute or reduce the properties provided by the clay mineral, i.e., plasticity, shrinkage, compaction, etc. This is particularly true if the non-clay mineral is present in relatively minor amounts (< 25 per cent). If they are present in larger amounts and particularly if the non-clay minerals are very well sorted, they may serve to determine the properties of the clay or soil. Thus in some soils with a high content of well-sorted silt sized particles, the clay mineral composition may be relatively unimportant in determining the physical properties.

It should be emphasized that in some soils and clays there is sufficient organic material and of a type that exerts an important influence on properties. Also, in some cases, water soluble salts may be present that can exert an influence on properties. In summation, the clay minerals are the essential components of argillaceous materials, they are not, however, the sole components. The other components must be considered in any evaluation of the causes of the properties of any particular clay.

Clay Mineral Composition of Some Typical Argillaceous Materials

Clays called kaolin are composed of extremely small particles of the kaolinite clay mineral frequently with minor amounts of small particle quartz,

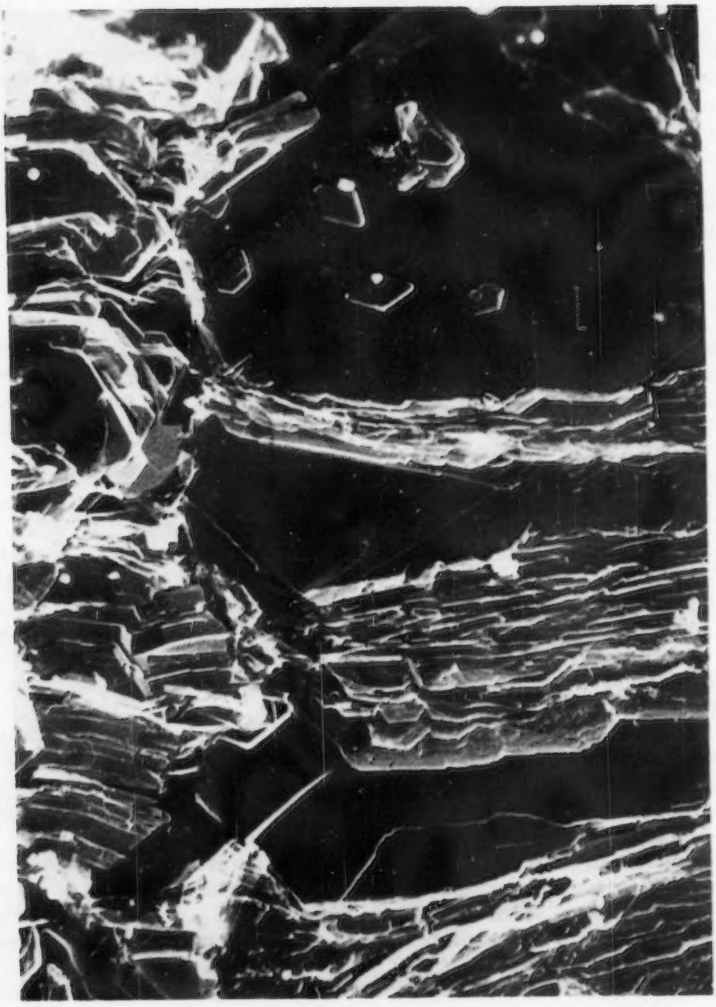

Fig. 7

feldspar, and mica. The kaolinite particles are often 1 to 10 micron in diameter. Each particle is like a small book, Fig. 7, made up of leaves 7A thick with the structure noted above. When the kaolin is stirred in the water the books separate from each other and some of the books may split by the separation. The amount of such splitting of books depends on the vigorousness of the dispersion.

Bentonites are composed of particles of montmorillonite frequently less than one micron in diameter. Often there is only a very slight non-clay mineral content. On dispersion in water the montmorillonite particles tend to cleave between the 10A units, so that the particle size distribution shown by

such a suspension frequently has no relation to the size of the montmorillonite particles in the dry bentonite.

Sedimentary clays may have any clay mineral composition.

Shales are composed of clay mineral particles of any size less than about 10 microns of any of the clay minerals. Frequently several of the clay mineral groups are represented. Illite and chlorite are frequent components of shales and they are frequently arranged with their large flake surfaces parallel to each other thereby forming the lamination of the shale. Various types of non-clay minerals are frequently significant components of shales. Mixed-layer structures are common in shales.

Weathered surficial material (soil of the geologist) may have any clay mineral composition (except attapulgite and sepiolite are very uncommon) and a large variation in non-clay mineral components. The clay minerals in such material are frequently relatively poorly ordered and of small particle size. Also mixed-layers are extremely common.

Origin of the Clay Minerals

The clay minerals can form under a variety of conditions and from various parent materials. They may be alteration products of other minerals or reaction products forming as a consequence of various natural processes as indicated in the following:

Weathering

Clay minerals form at or near the earth's surface as a consequence of weathering processes. The particular clay minerals that form vary, depending on the climate, topography, vegetation cover, parent rock, and time during which the weathering has continued.

With more than about 15 inches of annual rainfall and with at least moderate topographic relief, the dominant movement of water in the surficial material is downward to the water table. There is, as a consequence, a continuous leaching of the soils, which in initial stages involves the removal of alkalies and alkaline earths from the parent material. In the case of carbonates the calcium and magnesium are dissolved and the parent minerals disappear. In the case of silicates such as the feldspars, amphiboles, hornblendes, etc., this generally involves a break up of the original structure and a regrouping of components into the layer structures of the clay minerals. In detail this can be visualized under acid conditions as the penetration of the proton into the structure, joining with an oxygen thereby breaking a bond to free a cation. It may also be visualized as an oxidation reaction breaking the original structure, or as a cation exchange reaction.

The initial leaching of such minerals as muscovite, biotite, chlorite, and the clay minerals, illite and chlorite, tends to remove the cations from between the silicate layers. Thus the potassium is removed from between the sheets of illite, muscovite and biotite, and the magnesium is removed from the brucite layers of the chlorite. The removal of the potassium and magnesium tends to "degrade" the micaceous minerals, i.e., it tends to remove the "bridge" or "glue" which hold the silicate sheets together and as a consequence permitting water to enter between the sheets. The mica particles are therefore, more easily dispersable into smaller particles and their properties change, e.g., the Limit values go up.

The first stage in such degrading is a partial removal of the interlayer cations so that water can enter between only some of the layers. There is then a mixed-layer structure developed with water being able to penetrate between some layers and not between others. The leaching may be carried to substantial complete removal of the interlayer cations with water able to penetrate between all layers—such a structure is expandable and has the properties of montmorillonite, i.e., in weathering the micas can be degraded to montmorillonite.

As indicated the initial stage of alteration under leaching conditions is likely to degrade the micas and form expandable minerals. Further alteration, or weathering of a parent rock not containing micas and substantial amounts of alkalies and alkaline earths, varies depending largely on the dominant pH of the downward seeping water. With a continuous cover of vegetation producing organic acids by the process of decay, the waters are acidic (after the alkalies and alkaline earths have been leached) and the R_2O_3 compounds may be leached leaving behind a concentration of silica, e.g., in podsolic soils. When the climate is such that the organic material is rapidly oxidized at the surface, e.g., in a hot area with seasonal rainfall, the waters are likely to be about neutral or even slightly alkaline so that silica is removed concentrating iron and aluminum hydrates at the surface, e.g., in lateritic soils. In both the podsolic and lateritic soils, kaolinite may be an intermediate product between any degraded mica and the final weathering product.

As a consequence of alteration processes and water movement downward from the surface, the alteration products tend to develop in a series of zones or layers, i.e., a weathering or soil profile is developed. The process of alteration can only be revealed by the study of a series of samples from various zones in the profile, and not by single samples of parent rock and weathered material. Similarly a surface sample does not show necessarily the properties of a sample taken only a short distance below the surface.

Soil researches have shown abundantly that if a given type of climate prevails in a large area for a very long time, the same type of weathering product develops regardless of the parent material. There are, for example, large areas on the earth's surface of divergent types of bed rock which are covered by deep lateritic soils. It should be emphasized that in such large regions there are likely to be small local areas in which the soil although superficially appearing to be lateritic, has somewhat different components and properties. These are areas in which the influence of the parent rock hasn't been completely destroyed or peculiar conditions of topography and drainage cause differences. The detection of such local variation may be of very great importance in engineering activities and is often a very important responsibility of the geologist and clay mineralogists.

It should be emphasized that alteration as described above can take place very rapidly, particularly under warm moist climate conditions, e.g., in a matter of a few years, substantial clayey material may develop on fresh volcanic debris. Also the amount of alteration may be misleading—the writer has seen apparently fresh volcanic rocks which on X-ray analysis were found to be composed of more than 50 per cent montmorillonite.

In regions in which there is less than about 15 inches of rainfall annually, there is no dominant movement of water downward from the surface. As a consequence the alkalies and alkaline earths are not removed by leaching, but as a matter of fact may be concentrated in the surface material as a consequence of the upward movement of water because of large surface evaporation.

Under such conditions the minerals developing in the surficial material are illites, montmorillonites, and chlorites. If the concentration of magnesium is very high as it might be in a desert salt flat, attapulgite-sepiolite minerals may develop. Under conditions of very high sodium content, zeolite minerals may form.

Investigations of very cold regions indicate that there is substantially no chemical alteration of the surficial material under such climatic conditions.

Sedimentation

Weathered material may be washed into salt flats or lakes. It may be washed into rivers and then carried into lakes or to the sea. Essentially this means that weathered debris may be transported from one environment, e.g., fresh water into another environment, e.g., marine. Available evidence indicates that there may be a change in the clay mineral composition of argillaceous debris as it is carried from one environment to another (Grim, 1958).

There seems to be little change on passing from weathered debris into fresh water, e.g., rivers or lakes. On the other hand when passing from fresh to marine water there is a tendency for illite and chlorite to develop particularly from the degraded micas. The degraded micas take up potassium and magnesium from the sea water essentially reversing the events of the weathering leaching process. It is clear that in the geologic past this so-called diagenetic change has in some cases been substantial and in other cases trivial—the reason for this variation is not well known. In some cases apparently all of the argillaceous components of the debris, not only the degraded micas, have altered to illite and chlorite, whereas in other cases there appears to have been very little alteration.

Argillaceous debris carried into highly saline waters are likely to undergo substantial changes. Illite is favored by high potassium contents, moderate magnesium contents favor chlorite and large amounts favor attapulgite-sepiolite, high contents of sodium favor the formation of zeolites.

Metamorphism

Deep burial and the application of tectonic forces tend to squeeze out the water from between the silicate sheets and to increase the size and perfection of crystallinity of the micas. They tend, therefore, to change the expandable clay minerals (montmorillonite) and mixed-layers in the direction first of illite and clay mineral mica and then to muscovite, biotite, chlorite, etc. With very intense metamorphism the minerals may be completely dehydrated and changed over to non-hydrous crystalline compounds. It appears that kaolinite is not changed necessarily until the very intense phases of metamorphic alteration.

Hydrothermal

Hot gases and vapors derived from cooling igneous material or extremely deep meteoric circulations frequently cause substantial alteration of the rocks through which they pass. Thus, many veins of ore minerals which have been formed by hydrothermal waters are surrounded by an aureole in which the wall rock has been changed to clay material by the same water that deposited the vein material. Frequently there is a zonal arrangement of clay mineral alteration, with an inner zone of white mica followed in turn by kaolinite,

montmorillonite and chlorite. In some cases, several intervals of alteration are superimposed on top of each other. In other cases, a single clay mineral has resulted from such alteration which seems to be a reaction product of the hydrothermal agents and a given rock. Large masses of montmorillonite, e.g., on Ponza, Italy are examples of such mono-clay mineral alteration. The huge deposit of halloysite at Eureka, Utah is another example.

Such hydrothermal alteration accompanies the action of hot springs and geysers. Steiner (1953) has described such alteration in the hot spring and geyser area of the north island of New Zealand. Information on the nature and location of such alteration, is, of course, very important for construction activities in these areas.

All of the type of clay minerals have been described as hydrothermal alteration products.

A similar mode of origin of the clay minerals is that occurring in the vesicles in basic igneous rocks. In many cases such clay minerals are unquestionably due to the alteration of the enclosing basalt rock by the gases escaping from the basalt during its cooling. The gases forming the vesicles also formed the clay minerals. Montmorillonite sometimes with zeolites are frequent fillings of such vesicles. It is interesting to note that the solid basalt containing the vesicles often contains a considerable quantity of montmorillonite.

Bentonites

Bentonites are usually defined as clays composed essentially of montmorillonite that have been formed by the alteration in situ of volcanic ash. In such cases, it appears that the ash has fallen in a body of water and that the devitrification of the ash and the development of the montmorillonite has been bout contemporaneous with the ash fall. As engineers well know, such clays with their large content of montmorillonite have extremely unusual physical properties. The properties of bentonite vary from one occurrence to another depending on the composition of the montmorillonite and the adsorbed cation it carries which in turn appear to depend on the composition of the original ash and the water into which it fell.

It should be pointed out that there are clays that have formed by the alteration of ash which are not dominantly composed of montmorillonite, e.g., illitic alter ash in Switzerland (Hofmann, 1956), and halloysitic-allophanic altered ash in New Zealand. Also there are clays composed of montmorillonite and called bentonite which are hydrothermal alteration products and not alteration products of volcanic ash in situ.

Methods of Identifying the Clay Minerals

To be useful clay mineral analyses must be accurate, complete, and quantitative. Since small percentages of certain clay minerals, e.g., montmorillonite, may largely determine the properties of a soil, an analysis which fails to detect small amounts of such minerals is worse than useless, i.e., it may be misleading.

Many techniques are available for clay mineral identification. X-ray diffraction, dehydration characteristics and changes taking place on heating, (for example, differential thermal analysis), optical properties, infra-red absorption, staining tests, etc. Chemical composition is not a satisfactory

procedure alone, but along with other data, may be extremely helpful. Electron micrography showing the shape of the clay particles is helpful in showing elongate particles, but otherwise has limited diagnostic value.

If a clay or soil is composed of a single well crystallized clay mineral with little or no non-clay mineral component, any of the above methods may be adequate. If the sample is a complex mixture of minerals, X-ray diffraction is the best technique. In many cases the X-ray diffraction data must be supplemented by other procedures in order to get adequate positive identifications.

In the writer's laboratory, the following general analytical procedure is frequently used. An initial powder diffraction diagram is obtained to get an indication of the general character of the clay mineral composition. The sample is also examined microscopically to determine the approximate size and abundance of the non-clay minerals. Sometimes if the sample is substantially a single clay mineral, the foregoing is adequate.

Generally the preceding step is followed by the dispersion of the sample in distilled water without a chemical dispersing agent if possible (this is usually possible if the water soluble salts are removed by washing) followed by fractionation into various particle sizes in order to eliminate non-clay minerals and concentrate the clay minerals in certain sizes. The preliminary examination usually suggests at what particle sizes the fractionation should be made. Often the fractionation is at about 2 microns, 1 micron, and minus .5 micron.

Oriented aggregates (Grim, 1934) are prepared of the fractions containing the clay minerals by allowing suspensions to evaporate on flat surfaces. The flake shaped clay minerals are deposited flat, one on top of another, so that X-ray diffraction of such aggregates emphasizes basal plane reflections which are those most characteristic of the clay minerals. If the abundance of the non-clay minerals is not large, oriented aggregates of the whole sample are also prepared. Several oriented aggregates are prepared of each fraction and each sample.

X-ray diffraction spectrograms are obtained for the oriented aggregates without further treatment, after treatment with ethylene glycol which selectively changes the basal spacing of the expandable minerals, after heating at moderate temperatures (200° C) to collapse the expandable material, and after heating to high temperatures (450° - 550° C) to selectively dehydrate and change the structural characteristics of certain of the clay minerals. From all these X-ray diffraction data a fairly precise quantitative estimate of the clay minerals composition can usually be made (Johns, Bradley and Grim, 1954).

Sometimes the foregoing data are not adequate and additional procedures must be followed. For example, X-ray diffraction data obtained continuously while the sample is heated (Weiss and Rowland, 1956) may be necessary to unravel the nature of mixed-layer assemblages. Differential thermal analyses may be helpful in identifying kaolinite because of the intense and characteristic exothermic reaction shown by this mineral at about 950° C. Electron micrographs may be needed to confirm the presence of tubular halloysite. In some cases a potassium determination is necessary to evaluate the illite.

Sometimes when it is desired to study the expandable material in detail, the sample is treated with a potassium salt which causes the mineral to collapse after drying so that it will not expand with glycol treatment if it is degraded illite rather than montmorillonite derived from a bentonite. Treatment

with a magnesium salt seems to similarly differentiate degraded chlorite from montmorillonite derived from bentonites.

The most difficult problem currently in clay mineral analyses is to distinguish poorly organized kaolinite and chlorite in a complex mixture. Sometimes it cannot be done, and one must be content to designate a component as either kaolinite or chlorite.

Obviously from the above consideration, the quantitative clay mineral analysis of many clays and soils is a difficult problem that taxes the ability of persons with background and experience in such matters. It is also obvious that there is room for difference of results between experienced workers and for differences of opinion as to the names that should be applied to some clay mineral components.

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Fig. 1(a) Silica tetrahedron
(b) Sheet structure of silica tetrahedra

Fig. 2(a) Alumina octahedron
(b) Sheet structure of alumina octahedra

Fig. 3 Kaolinite structure

Fig. 4 Electron micrograph of halloysite showing tubular particles

Fig. 5 Montmorillonite structure

Fig. 6 Attapulgite structure

Fig. 7 Electron micrograph of kaolin showing kaolinite books.

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PHYSICO-CHEMICAL PROPERTIES OF SOILS: ION EXCHANGE PHENOMENA

A. W. Taylor[1]

FOREWORD

This symposium on the physico-chemical properties of soils was arranged under the auspices of the Committee on Mechanical Properties of Materials, of the Engineering Mechanics Division, sponsored jointly with the Soil Mechanics and Foundations Division.

Its objective was to summarize for civil engineers the current status of knowledge in this field and to stimulate further research and applications of this comparatively new and fundamental tool of soil technology.

The four papers in this symposium have been published as Proc. Papers 1998, 1999, 2000, and 2001.

ABSTRACT

The origin of the various types of electrical charges carried by soil colloids are described and the influence of different species of exchangeable cations on the forces arising from the electrical fields due to these charges is discussed in terms of the theory of the diffuse double layer. The principles of the application of this theory to the study of floculation and swelling phenomena are described and examples are given to illustrate the way in which it may be used to predict the effects of different types of exchangeable cation and salt concentration on the mechanical properties of soils.

Note: Discussion open until September 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 1999 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. SM 2, April, 1959.

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In any attempt to give a quantitative interpretation of the effects of the exchangeable ions on the physical properties of a soil system, it is first necessary to understand the nature of the colloidal material present and then to choose a suitable mathematical model to interpret the observed behavior.

The clay fraction present in a soil is usually defined as the part of the soil material which has an equivalent Stokes diameter of less than 2μ. This definition, although arbitrary, is useful because this size limit corresponds fairly closely with that below which surface properties begin to dominate the chemistry of the material. Also, although the larger particles may carry electrical charges on their surfaces, the amount when measured in terms of charge per unit mass—the cation exchange capacity—is small for particles whose size is larger than this: below it there is a tendency for the charge to increase rapidly with decreasing size, although there is no simple direct relationship between the two parameters.

Chemical and mineralogical analysis of the clay fraction after this has been separated from the bulk of the soil by sedimentation methods usually reveals the presence of two principal mineral entities which dominate the colloidal behavior of the system—the clay minerals proper and the iron oxides. In soils of temperate regions, where the clay minerals are the dominant components, it may easily be shown by electrophoretic measurements or ion exchange studies that there is a permanent negative charge carried by the colloidal material, and that this is associated with the clay minerals. The amount of this charge, which can be regarded as equal to the cation exchange capacity, depends upon the amount and type of clay mineral present; even in one single soil sample it may vary considerably with the pH of the system.

Electrical Charges Carried by Soil Colloids

In recent studies on kaolinitic clays by Schofield and Samson[8] it has been shown that the changes in the overall charge are due to the presence of three different types of charge which may be distinguished on the basis of the way in which they arise. The greater portion of the charge is due to isomorphous replacements of ions within the clay lattice; this is a permanent feature of the crystal and does not depend upon the composition of the ambient solution. The amount varies in different types of clay and its magnitude is approximately equal to the cation exchange capacity of the clay when this is measured in a dilute electrolyte solution (M/50 or less) buffered at pH 7.0. Under these conditions the contributions made by the other types of charges—which vary with the pH of the system—are usually small. Typical figures for the isomorphous replacement charge in the various types of clay are given in Table I.

Kaolins	3 - 8	meq/100 gram
Illitic Clays	40	" "
Montmorillonites	80	" "
Vermiculites	110 - 120	" "

Table I. Approximate isomorphous replacement charges carried by different types of clay.

When suspended in solutions with pH values above 7.0 - 8.0 all clays appear to develop extra amounts of negative charge due to dissociation of ≡ SiOH groups present at the edges of the tetrahedral layers according to the reaction

$$\equiv SiOH \rightleftarrows SiO^- + H^+ .$$

The amount of this dissociation increases with increasing pH and results in an increase in the cation exchange capacity which is of the same order of magnitude in all the clay minerals and causes an increase of the order of 5 - 10 meq/100 grams over the pH range 7 to 10. At pH values above 10 reliable measurements cannot be obtained because the clay then begins to suffer chemical attack by the alkali.

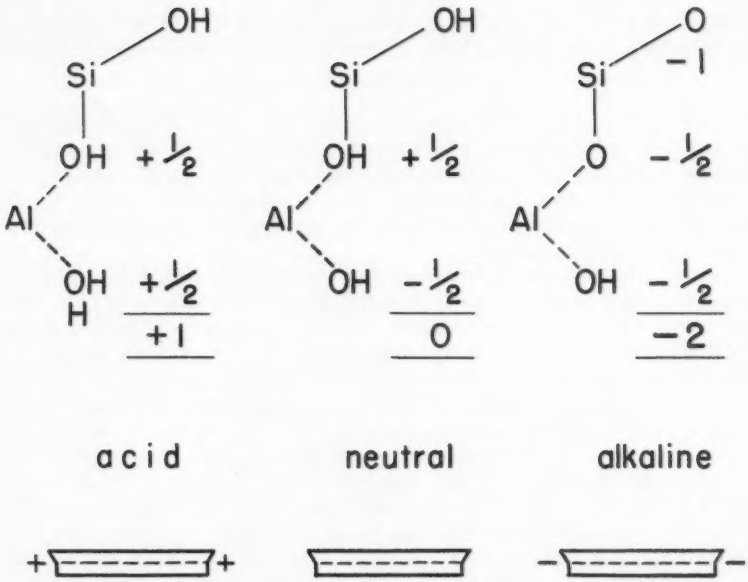
At pH values below 7.0 it has been demonstrated by Schofield and Samson working with kaolins and by Van Olphen[9] for montmorillonites that a decrease in the cation exchange capacity occurs which is not due to a change in the total amount of negative charge carried by the clay, but is due to the formation of positively charged sites by the adsorption of protons on the exposed edges of the octahedral layers. The particles thus become 'bipolar' carrying both positive and negative sites on the clay crystal. As the pH is reduced increasing amounts of positive charge are developed while the isomorphous charge remains unaltered, so that the overall net charge—which governs the cation exchange capacity—shows a progressive reduction with increasing acidity. The amount of this reduction is usually about 1 - 3 meq-100 gram for the kaolins, but may be larger in clays with finer crystals which have a larger proportion of exposed edges. The mechanism of the reactions which cause these variations of the charge carried by clays, and the resulting distribution of the charges over the platelike crystals are illustrated in Fig. 1.

The effect of these variations is much more marked in kaolinitic clays than in illites or montmorillonites; in the latter the isomorphous charge is large and dominant, while in the kaolins it is of the same order of magnitude as the charges produced by the adsorption and desorption of protons. Thus while in the kaolins the total charge may be increased several times over the pH range between 3.5 and 9.0, the same reaction in a montmorillonite clay will produce a change which is only a small fraction of the whole.

Studies of the clay fraction of soils as opposed to mineralogically pure clays reveal greater changes in charge than can be accounted for by these mechanisms we have discussed. The figures presented in Table II were obtained by Schofield in a detailed study of the electrical behavior of a Rothamsted soil.[7]

Comparison of the figures in the second column, which gives the negative charge (in milliequivalents/100 gram soil) carried by the untreated soil with those in the third column, which gives similar figures measured on the same soil after the iron oxide has been removed by treatment with a solution of N/5 ammonium oxalate buffered at pH 3.5, shows that the variation in charge with the pH of the soil is greatly reduced. Studies of the uptake of chloride ion by these samples indicates that the greater part of the change in the charge of the untreated soil is due to the formation of positive charges on the surface of the iron oxide at the lower pH values. The contribution of positive charge made by the proton adsorption on the edges of the illitic clay is clearly shown in the change of the overall negative charge of the iron-free material.

The presence of electrical charges of opposite polarity is of primary importance in regard to the understanding of the mechanical behavior of soil systems, since they are capable of exerting strong electrostatic bonding



The development of charges on the edges of Kaolin clay particles, together with the resulting distribution of charges on the particles.

Fig. 1

forces between the colloidal particles, when the effectiveness of the mutual repulsion between the permanently charged negative surfaces is reduced. These bonding forces are those responsible for the formation of the stable semi-rigid or plastic structures characteristic of flocculated clay systems. Where however there are no positive charges present or the effect of the negative charge—which is always the most prominent feature of these systems, as shown in Table II—is dominant, strong forces of mutual repulsion between the particles will come into play. Under these circumstances the clay will lose all rigidity and take on the character of a deflocculated liquid slurry with no mechanical strength. The most important factors which control the mechanical behavior of soil systems are therefore those which influence the effectiveness of the forces of repulsion between the negatively charged clay particles, and of these there are two—the type of exchangeable cation present in the clay and the salt concentration of the solution with which it is in contact. The role played by the positive charges is important but secondary except perhaps in

pH	Negative charge (meq/100 gram)	
	Untreated soil	Iron free soil
2.5	--	23.2
2.6	21.7	--
3.1	21.6	--
3.65	--	23.5
3.8	22.3	--
4.15	--	23.3
5.5	24.0	--
6.2	25.7	--
7.15	27.0	--
7.4	28.0	--
7.5	--	26.8

Table II. Cation exchange capacity of Rothamsted subsoil clay as a function of pH.

some soils of tropical regions where iron or aluminum oxides constitute the bulk of the colloidal material present. When the environment is favorable the bipolar nature of the particles will control the properties of the system, but this will only happen insofar as the effectiveness of the negatives charged is suppressed.

Influence of Exchangeable Cations

If two electrodes are introduced into a suspension of clay in water and a direct current is passed between them, the suspended clay will be seen to move towards the anode showing that the clay particles carry an overall negative charge. This implies that the exchangeable cations which balance the charge on the particles to preserve electroneutrality in the system as a whole are present as free ions in the liquid phase and are not held to the surface of the clay by any type of chemical bond. If this were so, the charge on the clay particles would be reduced to zero and they would show no electrophoretic movement. These cations are however attracted towards the negatively charged particles and form a diffuse layer of positive charge surrounding them in a manner analogous to the way in which the atmosphere of the earth is retained by gravitational forces. This picture is borne out by the observation that ion exchange reactions in soil systems are very rapid, being controlled only by the speed of ionic diffusion and the rate of mixing of the solutions used. Specific bonding forces between the ions and the surface do not seem to be important except in one or two special cases such as the selective "fixation of potassium" by certain types of clay.

A clay suspension can therefore be regarded as composed of negatively charged particles surrounded by diffuse envelopes of positive charge contributed by an excess of cations present in the solution: associated with this envelope of charge there will be a gradient of electrical potential whose intensity decreases with increasing distance from the surface of the particle according to the basic differential equation

$$\frac{d^2\psi}{dx^2} = \frac{4\pi e}{D} \sum z_i \, n_{i_\delta} \exp -z_i \, e\psi/kT$$

where ψ is the electrical potential at a distance x from the surface of the particle, z_i is the valency of an ionic species whose concentration is n_{i_o} in the equilibrium solution at a large distance from the particle surface where the gradient of potential is zero, D is the dielectric constant of the solution, e the electronic charge, k the Boltzman constant and T the absolute temperature. This equation expresses the relationship between the electrical potential at any point in diffuse layer and the volume density of charge at the same point. The latter is calculated by summing the contributions made by all the ionic species present.

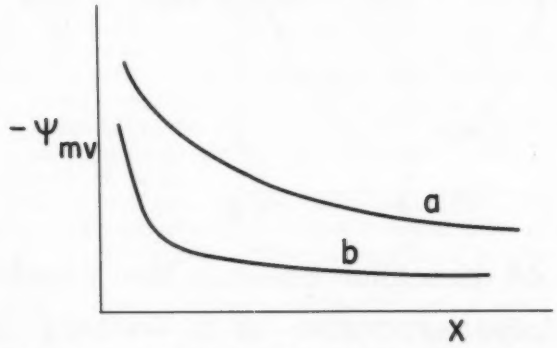
The complete integration of this equation to obtain the electrical potential as a function of distance is a matter of some difficulty except in the special cases where the electrolyte present is one of a simple symmetrical type such as NaCl or $CaSO_4$, but in dilute systems such as are usually present in soils the influence of the anions is usually small and the equations derived by Verwey and Overbeek[10] and by Kruyt[5] can be applied to give a satisfactory interpretation of the effects of cationic valency and electrolyte concentration on the behavior of soil colloids. The relation between potential and distance as obtained by Verwey and Overbeek takes the form of the equation

$$\exp.^{ze\psi/2kT} = \frac{(\exp.^{ze\psi_o/2kT}+1) + (\exp^{ze\psi_o/2kT}-1) \exp.^{kx}}{(\exp.^{ze\psi_o/2kT}+1) - (\exp^{ze\psi_o/2kT}-1) \exp.^{kx}}$$

where the symbols have the same significance as before, except that ψ_o represents the difference of potential between the solution and the surface of the particle, and k is the Debye-Huckel coefficient for the electrolyte solution, which is a function of ionic valency and concentration. The effect of changes in the valency of the ions present and of the concentration of the electrolyte as a whole on the structure of the diffuse layer as predicted by this equation are illustrated in Fig. 2, where curve (a) represents the situation present under conditions which favour extensive development of the diffuse layer: this is the case where the clay is in equilibrium with a dilute salt solution containing monovalent cations. If divalent cations are substituted, or the salt concentration is increased the potential at any given distance (x) from the surface will tend to fall, and the distribution of potential will tend to take the form illustrated by curve (b).

In applying this treatment to the calculation of the force of repulsion between two negatively charged particles separated by a distance 2d it is necessary to derive the expression for the potential curve in a situation illustrated in Fig. 3. Here curve (a) represents the type of relation which may be expected where the diffuse layers are well developed and strong repulsion occurs: curve (b) will represent the situation in more concentrated electrolyte solutions or in systems with polyvalent ions. The detailed derivation of a general mathematical expression relating the force of repulsion between two particles to the distance of separation, the electrolyte concentration and the type of exchangeable cation presents even greater mathematical difficulties than the integration of the fundamental equation. These are much too formidable to be discussed within the framework of the paper, but a very extensive discussion of the problem may be found in the treatise by Verwey and Overbeek.[10]

POTENTIAL DISTRIBUTIONS NEAR CLAY SURFACES CALCULATED FROM INTEGRATED POISSON-BOLTZMANN EQUATION

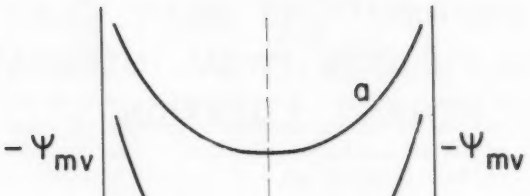


Distribution of potential as a function of distance from a negatively charged surface.

Fig. 2

The general character of the process may be stated in non-mathematical terms if we regard the diffuse layer of positive charge surrounding the negatively charged particles as a screen or shield surrounding them so that the normal force of repulsion between them is reduced. The efficiency of this screening process will depend upon the distribution of cations in the diffuse layer of positive charge, and this will in turn depend upon the properties of the ions themselves. Divalent ions are more strongly attracted towards the negative surface than monvalent and therefore form a thinner layer with a higher local density of charge: this has a more effective screening action so that the forces of repulsion between the negative charges are less effective even when the particles are close together. Replacement of divalent by monovalent ions reduces the effectiveness of the screen so that the particles repell each other over greater distances. The effect of increased electrolyte concentration is more difficult to describe in these terms, but it has a similar effect in that it results in a contraction of the space charge into a smaller volume so that it acts as a more effective screen.

Although the valency of the ion is the dominant factor which controls the structure of the diffuse layer, this is also affected by other parameters such as ionic size, polarizibility and hydration, and any complete mathematical picture of the double layer must take these into account. From an empirical point of view however a clear relation exists between the efficiency of an ion as a flocculating agent and the difficulty experienced in displacing it from the diffuse layer by other ions. The less easily replaced polvalent cations produce the most completely flocculated systems, and soils which contain these show the most stable mechanical behavior.



Distribution of potential between two negatively charged surfaces separated by a distance 2d.

Fig. 3

Determination of the Exchangeable Cations

It is therefore clear that information about the exchangeable ions contained in a soil may be of considerable help in predicting its mechanical behavior. In the great majority of cases this information can easily be obtained from a simple quantitative experiment in which a weighed sample of dry soil is leached with a suitable salt solution so that the cations removed from the soil may be determined by chemical analysis of the final extract.

The choice of a suitable reagent for such an experiment is governed by two principal considerations. It should not contain a cation which is likely to be already present in the soil itself, and it should be the salt of a cation which is strongly adsorbed by the soil so that it will be efficient in displacing the ions already present. The order of firmness with which exchangeable cations are held is represented by the well-known lyotropic series:

Li: Na: K: NH_4: Rb: Cs

Mg: Ca: Sr: Ba: Al.

$\longrightarrow$

increasingly strong adsorption.

This order of retention is approximately the same for most clays, although occasional changes do occur in some types: there is also some variation in the relative position of the two monovalent and divalent series.

The most common exchangeable ions found in clay soils of temperate regions are calcium, magnesium, potassium and sodium, of which the first two are usually dominant except in alkali or saline soils. The ions most usually chosen to replace these for estimation in exchange experiments are ammonium or barium, neither of which occurs in significant quantities in nature. Ammonium is the most convenient of the two since it does not cause any

interference effects in the chemical methods available for the determination of calcium, magnesium, potassium or sodium which can be readily carried out by flame photometric or spectrographic methods. It is very often employed in the form of a normal solution of ammonium acetate, buffered at pH 7.0, a very convenient reagent which allows routine measurements to be made under conditions of controlled acidity. The experiment can also easily be extended to measure the total amount of ammonium taken up so that an estimate of the total exchange capacity of the soil can be made. When this is done it is sometimes found that the total number of equivalents of cations removed is not equal to the amount of ammonium adsorbed: a deficit of cations—particularly in acid soils—may indicate the presence of some other ion such as aluminum which is present in addition to those estimated. An excess of cations removed over the ammonium taken up indicates the presence of excess salt occluded in the soil.

The chief value of information of this kind is that it may give a warning of dangerous conditions which may develop in a soil which is otherwise apparently satisfactory. The presence of large amounts of monvalent ion (more than 15 - 20% of the total)—and particularly of sodium ion should always be regarded as a danger signal. The danger is particularly acute in such cases if the soil is being maintained in a flocculated condition by the presence of excess occluded salt. As long as this remains little change may take place, but if the salt is removed by percolating rainwater or by irrigation with salt free water, the soil will deflocculate with a marked deterioration in strength and structure, changing from a material with a yield point (in the sense that it shows a limiting stress below which no continuous flow takes place) to a viscous slurry which will flow under stress conditions where the flocculated material is stable.

This process can easily be reproduced under controlled conditions in the laboratory, and the critical electrolyte concentration at which deflocculation occurs in a given sample can be found with some precision as shown in the work of Emerson[3] and Quirk.[6] In soils where the monovalent ions are less than 10% of the total present, which is almost always the case in soils which have not been flooded with saline water, the flocculating efficiency of the divalent cations is such that these soils are stable unless the salt concentration of the percolating solution is less than about N/100. This is rather less than the salt concentration present in the field, where the soil is not subjected to the disruptive tendencies produced by drying, sieving and rewetting. This condition therefore represents a stable situation.

As the proportion of exchangeable sodium in the soil increases the limiting concentration of electrolyte rises rapidly to a level of about N/5 when it is nearly saturated with the monovalent ion: this however represents a rather extreme situation which is very rarely found under field conditions. Owing to variations of texture and composition in natural soils it is very difficult to lay down any absolute standard of safety: the figure of 15% exchangeable sodium has been found useful by the U. S. Salinity Laboratory in connection with the stability of soils under irrigation.[11] As a general rule it may be said that any soil which contains more than this proportion should be regarded with suspicion and its behavior thoroughly evaluated by suitable testing.

Visual inspection of the soil in the field is inadequate. Owing to differences of salt concentration and texture within the soil profile, deflocculation is a more gradual process in the field than under the controlled conditions of the laboratory, and the symptoms can easily be overlooked. The first stages are

marked by movement of deflocculated clay into the natural drainage channels of the soil: if this process continues these become blocked by the clay so that the soil becomes impermeable to percolating water or solution, even though the moisture content is high. Re-establishment of normal drainage in a soil damaged in this way can be a very difficult and expensive matter.

Swelling and Compressibility

The effects of cation and electrolyte concentration on the forces of repulsion between clay particles have been clearly demonstrated experimentally by Bolt and Miller[1,2] in their measurements of the swelling pressures and compressibility of soil clays.

In a deflocculated sample of soil compressed in a cell with a semi-permeable membrane which allows the passage of water or electrolyte solution the resistance to compression offered by the clay will be found to increase as water is lost and the distance separating the particles is reduced so that the mutual forces of repulsion are increased.

If the system is regarded as built up of parallel charged plates, this repulsive force may be shown to be related to the osmotic pressure existing between the solution at the midpoint between the two surfaces (at distance d in Fig. 3) and the outer solution; where the electrolyte present is of a symmetrical type this pressure is given by the equation

$$P_s = c_o RT \left(c_m / c_o + c_o / c_m - 2\right)$$

where R is the gas constant, c_m the median electrolyte concentration and c_o that in the external ambient equilibrium solution. Since the value of c_m cannot be found experimentally it must be calculated from the equation

$$z_i \sqrt{\beta c_o}\,(x_o + d) = 2\sqrt{c_o/c_m} \int_{\phi=o}^{\pi/2} \frac{d\phi}{\sqrt{1 - (c_o/c_m)^2 \sin^2 \phi}}$$

where z_i is the valency of the ions, β is a constant depending upon the temperature, x_o a constant related to the surface density of charge carried by the clay, and ϕ a variable related to c_m. Solutions of this expression giving the swelling pressure as a function of particle separation and electrolyte concentration have been tabulated by Bolt,[1] who applied them to the study of the swelling of sodium saturated samples of illite and sodium and calcium saturated samples of montmorillonites. It was found that the shape of the compression curves obtained was in substantial agreement with that predicted by this expression, and—in the case of the illite—the influence of increased electrolyte concentration was also as expected. The disagreement between the absolute values of the observed and the expected swelling pressures was regarded as due to errors in the value used for the specific surface area of the clay and to the failure of the particles to pack under compression in the simple geometrical arrangement assumed in the mathematical model.

These experiments, as well as others by Hemwall and Low[4] and by Warkentin, Bolt and Miller[12] serve to illustrate very forcibly the influence of electrolyte concentration on the mechanical properties of clay soils. These differences are much more evident in soils which contain montmorillonites and show marked swelling than in illitic materials. The total expansion

observed by Bolt in a sodium montmorillonite suspended in M/1000 NaCl over the pressure range 0.1 - 100 atmospheres was about six times larger than that for an illite in a similar environment. Saturation of the montmorillonite with calcium ion reduced the swelling to approximately 40% of that in the sodium clay; no figures are available for the compressibility of a calcium illite, but it is clear that if the reduction is of equal magnitude it is likely to be very low particularly where the electrolyte concentration is appreciably higher than the level of M/1000 used in these experiments. This will always be the case in the solutions present in natural soils.

The difference in the swelling behavior of montmorillonite and illite clays is a result of the differences of the amount of surface area over which the repulsive forces are free to act: in the illitic material the expansion is due to repulsions between individual clay crystals which are stable units. In montmorillonites the forces act between the actual alumino-silicate layers of which the clay is built up. This produces interlayer-swelling in addition to the more limited inter-particle swelling of the more stable systems.

CONCLUSIONS

Owing to the wide variations found in the texture, composition and geological history of natural soils, it is hardly to be expected that any single theoretical treatment is likely to give a universal interpretation of the influence of chemical factors on the mechanical properties of soils in general. Nevertheless it is evident from the results obtained where detailed investigations have been carried out individual cases that the Gouy-Chapman theory of the diffuse double layer provides a satisfactory basis for the study of many types of clay soils, and may be used to derive useful information about probable mechanical behavior from the results of chemical tests.

This theory is most appropriate to soils which contain appreciable amounts of clay minerals in the colloidal fraction of the soil: where this is amorphous or contains large amounts of materials such as iron and aluminum oxides, which may carry positive charges so that the soil may show an isoelectric point of zero charge, less satisfactory results may be obtained because the nature of the colloidal material is too complex. Systems of this kind can however be easily identified during the course of the mineralogical and chemical tests which should always be conducted on any soil whose properties are likely to be of importance in connection with any engineering enterprise.

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Proceedings of the American Society of Civil Engineers

PHYSICO-CHEMICAL PROPERTIES OF SOILS: SOIL-WATER SYSTEMS

I. Th. Rosenqvist[1]

FOREWORD

This symposium on the physico-chemical properties of soils was arranged under the auspices of the Committee on Mechanical Properties of Materials, of the Engineering Mechanics Division, sponsored jointly with the Soil Mechanics and Foundations Division.

Its objective was to summarize for civil engineers the current status of knowledge in this field and to stimulate further research and applications of this comparatively new and fundamental tool of soil technology.

The four papers in this symposium have been published as Proc. Papers 1998, 1999, 2000, and 2001.

ABSTRACT

The heat of wetting, the diffusibility of hydrogen and isotopic prothium-deutrium exchange, indicate that a part of the water in a clay-water system should be considered as belonging to the mineral phase. The minerals thus consist of a central silicate skeleton with an outer part of aqueous composition. The presence of this outer part is the essential characteristic of the mineral phase in a clay-water system. Typical clay properties, such as cohesion and plasticity, are discussed in terms of this concept. The Bowden friction theory is assumed to be valid in clay-water systems.

Note: Discussion open until September 1, 1959. Separate discussion should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2000 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. SM 2, April, 1959.

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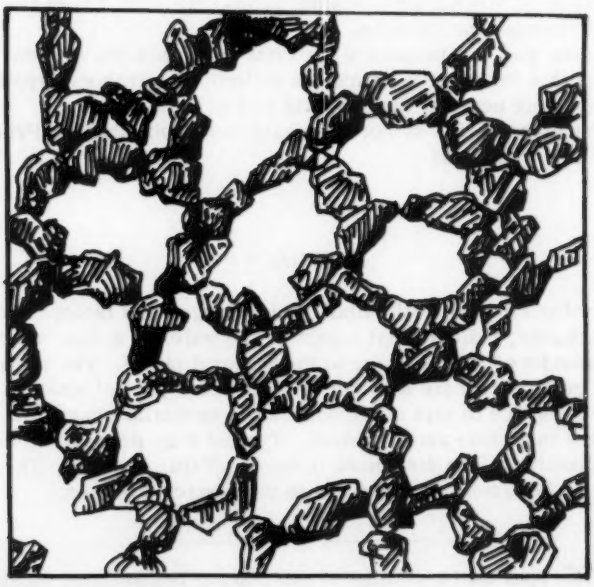
General Review

The mechanical properties of the so-called "cohesive soils" have been dealt with as a subject of scientific research for more than thirty years. In the early "twenties" the general opinion concerning cohesion was that it was due to some undefined action of a hypothetical amourpheous clay substance.

In 1923 Assar Hadding proved by X-ray analyses that clays had mainly a crystalline nature.

In 1925 Karl Terzaghi published his famous book "Erdbaumechanik auf Bodenphysikalischer Grundlage." Here we may find a remarkably modern discussion concerning the structure and bonds in cohesive soils. Terzaghi disagreed with the general opinion of cohesion at that time and drew attention to the adhesion between minerals, which he called "Null-reibung," and held that the cohesion in clays was due to adhesion between adjacent minerals. He suggested that the minerals stick to each other at the points of contact with forces sufficiently strong for the building up of a honeycomb structure, so that comparatively large amounts of water could be enclosed within voids built up of aggregates of minerals glued to each other by the adhesion forces. Thus, each cell in the honeycomb structure was supposed to be made up of numerous single mineral grains. (Fig. 1).



The Terzaghi-Casagrande concept of honeycomb structure, redrawn from Terzaghi: Erdbaumechanik auf bodenphysikalischer Grundlage.

Fig. 1

V. M. Goldschmidt, known mostly because of his geological and geochemical work, also published eight papers concerning clays. He was mainly interested in the clays as mineralogical raw materials, but dealt with their mechanical properties too. He performed a series of experiments with mixtures of clay minerals and various liquids such as water, benzene, carbontetrachloride, liquid sulphur dioxide, and ammonia. In this way he discovered that the mixtures had plastic properties, provided the liquids were of a certain character. If the liquid involved was non-polar, as for instance carbon-tetrachloride, he obtained mixtures without plasticity, analogous to what is found in mixtures of silt and water. He concluded that the clay properties (mainly the plasticity) were dependent upon both the crystal chemistry of the mineral phase and the atomic structure of the liquid phase.

In 1926 this view was presented in its conclusive form. Goldschmidt expressed here the opinion that the clay properties were due to crystalline minerals surrounded by a film of adsorbed water molecules and that the water molecules stuck to each other and to the minerals because of their dipole moment. In his published university lectures, he expressed the opinion that the flaky minerals in the highly sensitive clays are arranged in unstable cardhouse structures. Goldschmidt maintained that the surplus water was enclosed in the space between a few mineral flakes leaning upon each other and that the difference between clays of high and low sensitivity was due to a denser arrangement of the minerals in the clays of low sensitivity. This view is, therefore, somewhat different from the honeycomb structure of Terzaghi.

In 1932 Arthur Casagrande presented a theory of honeycomb structure in sensitive soils, very similar to that of Karl Terzaghi.

In 1946 the present author showed that there could be no important difference in the mineral arrangement of normally consolidated marine clays. With a given grain distribution, the sensitivity was independent of the water content. It was shown that sensitivity in quick clays was connected with the colloid chemistry of the system. For heavily overconsolidated clays of low sensitivity, Skempton and Northy in 1952 expressed the opinion that their low sensitivity was due to a collapse of all the structures during the consolidation.

In 1953 William Lambe presented a paper concerning the "Structure of Inorganic Soils." He strongly held colloidal chemical viewpoints and showed schematic diagrams of the mineral arrangements in clays. For the undisturbed marine clays an open structure similar to Goldschmidt's cardhouse structure is shown, whereas in fresh water sediments the structure was thought somewhat denser and in the remoulded clays was supposed to have a high degree of parallelism and short range order between the flakes. (Fig. 2).

A very similar view of the structure of the clays was presented by the author in 1955 and by Bjerrum and Rosenqvist in 1956. Even though indirect methods have given us very good reason to suppose that the mineral arrangement in the undisturbed sedimented clays is of the type suggested by Lambe, we have not had until recently sufficiently good means to investigate this point directly. Using a special technique described in his 1955 paper, the author could obtain a fairly good picture of the mineral arrangement based on optical microscopy, but these pictures were too fluffy to be sufficiently convincing.

In 1957 T. K. Tan presented a schematic picture (Fig. 3) of clay mineral network dominated by contacts between a corner of one mineral and the plane of another.

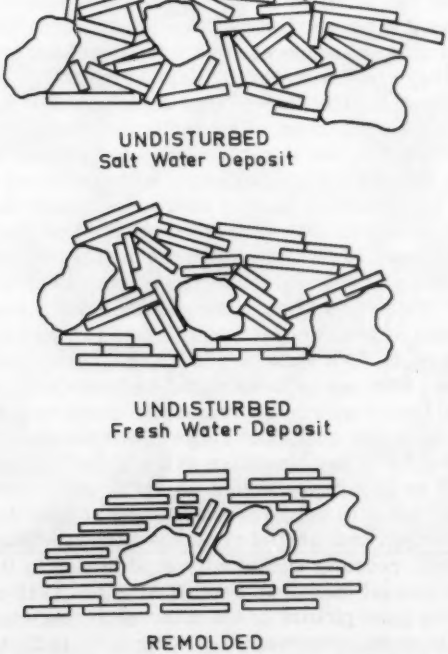
As will be shown later in this paper, we have now by means of electron microscopy been able to prove that the mineral arrangement in the undisturbed

marine clays really corresponds to the cardhouse structure suggested by Goldschmidt and later by Lambe, and in fact exactly corresponds to the imaginative drawing of Tan. The fact that this structure has been proven suggests in itself that the concept of friction and cohesion in clays should be re-examined. These two terms have been used ever since the days of Coulomb at the end of the 18th century. Today we speak of Coulomb's friction coefficients in terms of the effective stresses:

$$S = (\sigma - u) \cdot tg\varphi' + c'$$

σ = total stress
u = pore water pressure
φ' = angle of shearing resistance
c' = the c-value

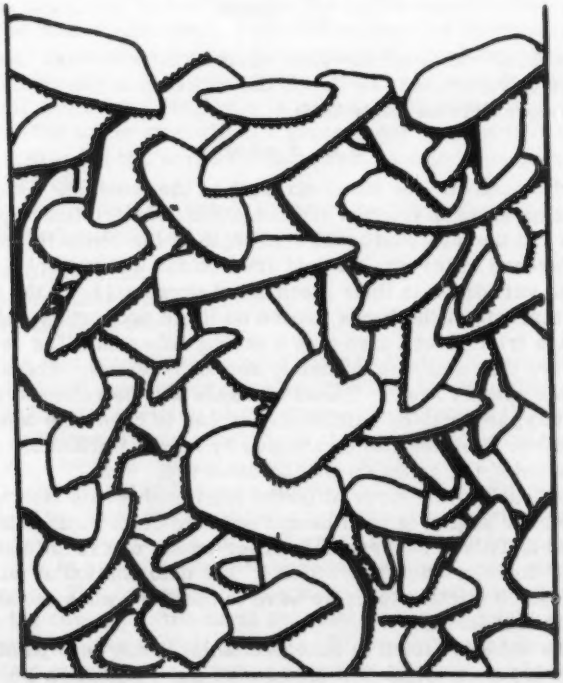
With most soils, independent of whether they are designated frictional soils or cohesional soils, we find that the energy of deformation at failure is different in the natural state and in the remoulded state, as long as the water content is kept constant. This is the case even when the remoulded soils are kept for a long time after the deformation. Of greater immediate importance than the energy of deformation at failure is, however, the shear strength



The Goldschmidt-Lambe concept of cardhouse structure, redrawn from T. William Lambe: The structure of inorganic soils.

Fig. 2

(drained or undrained). This value is normally lower in the remoulded than in the undisturbed state. The conventional concept of shear strength as made up of two terms, viz. the friction and the cohesion, gives us a very simplified picture of the complicated factors cooperating in what we call "the shear strength of a soil." The friction term is undoubtedly made up of different parts. We have the so-called "macro-dilatency," which is caused by the lifting of one mineral grain on the one side of the shear plane when the shear movement takes place. The so-called "micro-dilatency" is of corresponding nature, in which the resistance towards shear is due to the roughnesses in one mineral particle which must be lifted up and passed over the roughnesses of another mineral particle. It is obvious that both these terms will be proportional to the effective normal stress in the shear plane. Further, it is obvious that the resistance towards movement in such systems will depend upon the velocity of deformation and the viscosity of the liquid phase of the voids as deformation will involve a transport of the liquid phase. However, we have at least one additional term which gives shear resistance as a function of the normal stress in the shear plane. This term is called "the non-dilatent friction term." Here we understand resistance to shear due to



Schematic picture of clay ne
(T. K. Tan, 1957).

Fig.3

mass forces between the atoms in the one mineral and the atoms in the other mineral. Such forces will exist independent upon whether the minerals behave as completely elastic bodies or as unelastic, plastic bodies. In both cases the area of contact will increase and the distance between the atoms in two neighbouring minerals will decrease by increasing normal pressure. Due to these decreasing distances, some of the attractive forces will increase. Therefore, even the dilatent terms are, in fact, functions of the non-dilatant terms.

F. P. Bowden (1954), together with several co-workers, has carried out fundamental research concerning the nature of the non-dilatent friction. From his publications we may draw several important conclusions: primarily, that the area of actual contact between two solid bodies is very low, e.g. under normal loads the area of contact between two steel surfaces will be of the order 1/100 000 of the apparent area of contact, and that the real area of contact is nearly independent of the total area and only dependent upon the load. The real area of contact is only to a small extent influenced by the roughness and shape of the metal surfaces. The real area of contact may, as an approximation, be expressed by the following equation:

$$A = \frac{W}{Pm}, \text{ where}$$

"W" is the total load and "Pm" the yield value of the material. As the non-dilatant friction is regarded to be due to the adhesion in the area of contact, we may as an approximation state that:

$$F = A \cdot s, \text{ where}$$

"F" is the friction and "s" the shear strength of the junction. Thus we obtain a close connection between friction and adhesion. The friction represents the shear strength and the adhesion tensile strength of the bonds in the contact area. The difference in the non-dilatant friction for various solid substances is mainly due to variations in their mechanical properties, as the contact area as well as the strength of the bonds depend on these properties. Materials which give a high friction will also give a strong adhesion. The reason why we do not observe this adhesion is mainly due to the elastic stresses occurring in and around the contact areas. These stresses will be released when the load is taken away, and may be sufficiently strong to break the bonds of adhesion. When these stresses can be avoided by re-crystallization or by ductility at the junctions, the adhesion will be observed.

It seems as if each of the three different frictional terms mentioned will depend upon specific angles of friction and specific deformations at failure. The deformation at failure decreases in order from "macro-dilatant" through "micro-dilatant" to "non-dilatant friction." The deformation at failure will thus determine which friction term we have to consider when the shear strength value is measured.

Turning to the cohesion term in cohesive soils, it has been pointed out by the author (Fourth Intern. Conf. Soil Mechanics and Foundation Engineering, Vol. 3, p. 257) that the liquid limit of clays depends mainly upon three factors:

(1) The amount and nature of the minerals.
(2) The degree of electro-chemical saturation.
(3) The polarizability of the adsorbed ions.

There is an important difference between the montmorillonitic clays and the illitic clays with respect to the influence of the adsorbed cations. Thus lithium and sodium montmorillonites have abnormally high swelling power, because these cations cannot keep the single sheets of the lattice together. With the larger alkali ions, viz. potassium, rubidium and caesium, the internal swelling property is absent. Montmorillonites which have adsorbed the larger cations thus behave analogous to the illites, and give a regular increase in the liquid limit with increasing polarizability of the adsorbed cations.

Already Karl Terzaghi had noticed the influence of the different electrolytes upon the consistency of clays. In fact even the pre-historic ceramists seem to have known the influence of soda ash upon the consistency of clay when they manufactured their pottery. This influence is related to the degree of electro-chemical saturation. By leaching experiments, it has been shown that the electrical charge of the mineral grains may have an important increase during leaching due to dissociation of the adsorbed cations from the clay minerals. Correspondingly, the negative charge of the mineral may increase when certain anions are adsorbed as for instance a carbonate ion from soda ash or other anions as phosphates and silicates. Salts of such anions are used as dispersents in most soil laboratories. The effect is due to an increase in the repulsive forces between the highly negative charged minerals because of the anions adsorbed. This will change the mechanical properties of a clay water mixture at a given water content. We have shown (opt. cit. 55 p. 77) that the liquid limit of a Norwegian illite clay may decrease from 42 to 29 when small amounts of $Na_4P_2O_7$ is added. With addition of such salts, we may influence the shear strength at a given water content in a manner similar to that of leaching out the normal salt content; we may even simulate the formation of quick clays.

In addition to the factors acting in the remoulded state, the shear strength of an undisturbed sedimented clay is a function of

(4) The water content.
(5) The stress history.
(6) Diagenetic cementation.

To take the last point first, many soils are found where amorphous or crystalline mineral substances have precipitated between the minerals thus forming a continuous skeleton of minerals. It is obvious that shear through such a soil will depend upon a failure in the cementing substance or the original minerals. (See N. J. Denisov and P. A. Rebinder: On Colloid-chemical Nature of Cohesion in Argillaceous Rocks, Doklad op Acad. Nauk USSR, Vol. LIV, p. 519-522). Even though it is impossible at present to calculate quantitatively the real strength of a mineral crystal from the lattice forces of the substance, it is in principle easy to understand why a mineral crystal has shear strength. The cementation in addition to the effect of stress history is involved in the concept "structural cohesion" as mentioned by Vialov and Reltov (Fourth Int. Conf., Vol. I, p. 17). Disregarding the cementation which is unimportant in geologically young sedimented clays, we have to deal with friction and cohesion in a discontinuous system where the different minerals may be separated by a limited layer of adsorbed water between the grains, or where the grains are in physical contact.

Recent Investigations

The Structure of Clays

The Goldschmidt-Lambe theory of the mineral arrangement in undisturbed clays needed for its confirmation a technique not developed at that time. Using modern electron microscopy methods pictures as seen in Fig. 4 are obtained. As will be seen the arrangement tallies with the schematic picture in Lambe's paper and to an astonishing degree with the hypothetical scheme of Tan (Fig. 3). It is our opinion that the specimen really represents the mineral arrangement in an undisturbed clay. The specimen was prepared by quick freezing of a 2 cm thick plate of undisturbed marine Oslo blue clay, which was immersed in liquid air. Subsequently the frozen specimen was dried in vacuum at a pressure of about 10^{-3} mm Hg. The freezing in liquid air involved a very slight volume increase (3%), whereas the drying took place without any measurable change in dimensions. The dried plate was broken mechanically and the central part of the (vertical) surface thus obtained was used for the carbon replica, which was photographed in the electron-microscope.

The technique used for the preparation of the carbon replica is a slight modification of the technique published by Comer and Turley (1955). Because of the roughness of the surface of the freeze-dried and broken clay specimens, these were rotated (ca.400 rpm) during the evaporation of the carbon. The evaporation was performed at an angle of about 45° in order to get a relatively uniform film thickness. On top of the carbon film a thick (0,1-0,2μ) aluminum film was evaporated. This stage was introduced to avoid the influence of the polystyrene (applied hot on the specimen) on the dissolution of the clay in hydrofluoric acid, as this type of clay, contrary to most other types, is difficult to dissolve when the polystyrene is applied directly on to the carbon film. Before dissolving the clay, most of it is washed off with water. After the treatment in hydrofluoric acid, the replica is washed and mounted in the usual way.

The minerals seen on the photographs are mainly hydrous mica (illite) and 2 larger feldspar grains at a slightly inclined angle to each other. An important point seen in the stereopictures is the very open mineral arrangement dominated by contacts between corners and planes. We have found the same arrangement in all marine clays examined. Thus, the cardhouse arrangement is very well confirmed.

Influence of the Surface Ions

In Publication No. 9 from Norwegian Geotechnical Institute, pp. 111-112, it was stated that the shear strength of certain clays substituted with different cations had a relation to the calculated polarizability of the adsorbed cations. It was mentioned that, provided that Van der Waal's forces acted between the adsorbed cations on one mineral and the lattice field of an adjacent mineral, the forces should be proportional to the first power of the polarizability. As long as the forces took place between the same type of ions on the two minerals, the attraction should be proportional to the second power of the polarizability of the ions. In the case of illitic clays substituted with alkali cations, the first power relation was found. This is also to be inferred from the Goldschmidt-Lambe cardhouse arrangement, where edges of one mineral are in contact with the planes of another mineral, provided the conditions for adsorbtion are different along the edges and along the planes. Later experiments

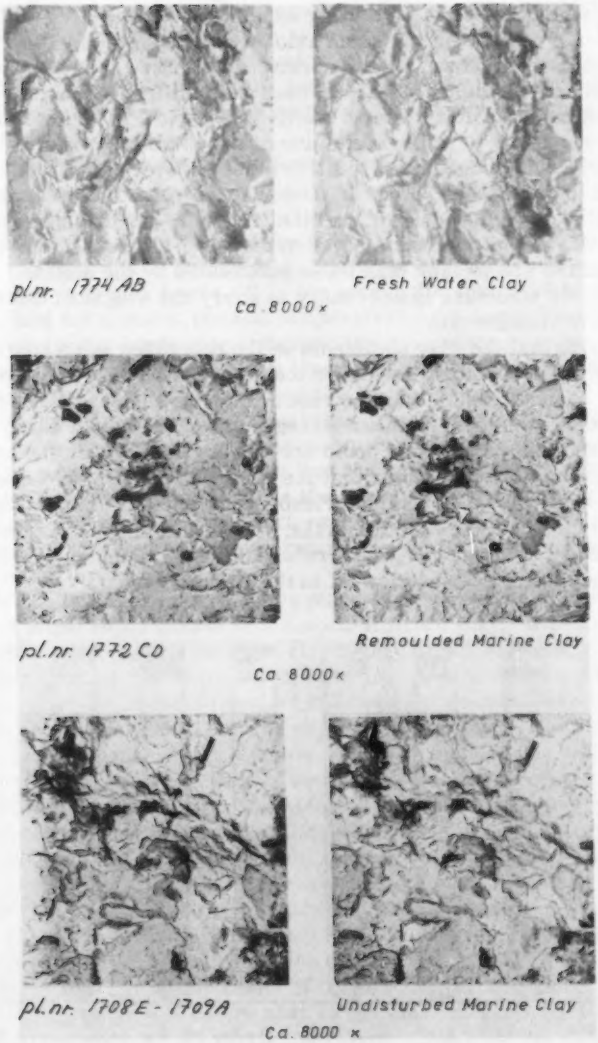


Fig. 4. Stereo micrographs showing clay structure obtained with electron microscope.

published at the Fourth International Conference confirm this concept. In discussions with colleagues the question has been raised whether the depolarizability or merely the size of the cations is of importance. This question seems to be elucidated by the following experiment.

In a series of test tubes, the same amount of a diluted clay slurry was mixed with the same amount of half-normal solutions of the chlorides of the following cations: lithium, potassium, caesium, aniline, cyclohex-cylamine. The final sedimented volume after 6 weeks is seen in Fig. 5. In this case we have the aniline and the cyclohexcylamine of very similar structure. Due to the conjugated double bonds of the aniline ions, however, this ion is much more polarizable than the cyclohexcylamine. The volume of the aniline clay is seemed to be about 50% larger than the volume of the cyclohexcylamine clays. This shows that the bonds between the minerals substituted by the cyclohexcylamine are weaker than those substituted by the aniline. Consequently, a denser structure is necessary to carry the weight of the minerals in the cyclohexcylamine clay.

If we suppose that the clay sediments in the test tubes were consolidated to the same water content, it seems obvious that the lithium clay would exhibit a lower shear strength than the rest of the clays, and that the aniline clay would exhibit the highest shear strength value. In such a case, the mineral skeletons and the water phase could not be very different in the single sample, the main difference being that the adsorbed cations on the surface of the minerals have very different polarizabilities, thus creating variations in the attractive force between the minerals. This again means that the adhesion between two neighbouring minerals with the same contact area is greatest in the aniline clay and lowest in the lithium clay.

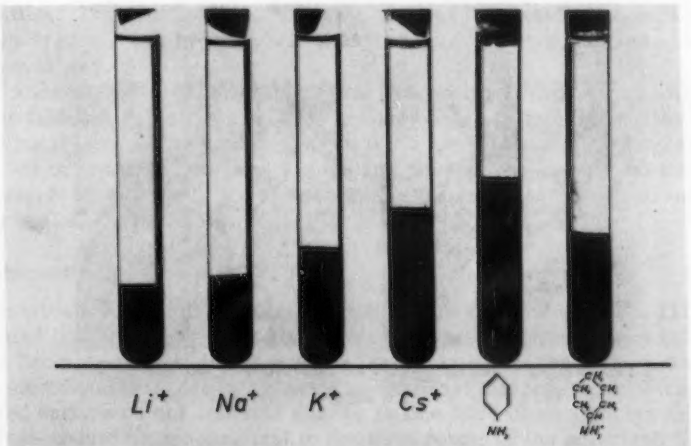


Fig. 5. Final sediment volume of equal amounts of clay minerals in water containing chlorides

Influence of the Water Phase

In order to understand the properties of clays, it is absolutely necessary to understand the properties of the water phase—or more correctly—the properties of the liquid phase of the systems. Although we know that the liquid phase does not consist of chemically pure water, we still continue to speak about the water phase, as the properties of this phase are so overwhelmingly influenced by the aqueous parts of the phase and only to a minor extent by the dissolved ions.

Everybody is familiar with the three normal aggregate states of water and knows that water has several peculiar properties compared to related chemical compounds: for instance, the melting and boiling points of water are abnormally high. The same is the case with the heat of melting and evaporation. The most characteristic feature of water is the existence of a high but variable number of hydrogen bonds. It is because of the hydrogen bonds that water is a liquid and not a gas at normal temperature. As small changes in the number of hydrogen bonds may influence the properties of water very much, it is obvious that the water of a two phase colloidal system, such as clay, will not be identical to the properties we know in macro-scale.

Experiments carried out on immersing dry clay powder in water show considerable heats of wetting. In the same way investigations of the vapor-pressure of moist clays have shown that the energy of water next to the minerals must be considerably lower than the energy of normal water at the same temperature. Furthermore, the difference in intrinsic energy between "clay water" and normal water, at temperatures above the freezing point, depends upon the temperature.

In order to investigate what takes place in the freezing of clay, we have performed some experiments with a normal illitic clay. It is well known that a considerable part of the water in clays does not freeze at 0° C. The mechanism of the freezing is, however, still obscure and is complicated by the fact that the pore water of clays does not consist of pure distilled water, but of dilute electrolyte solutions. At the same time the forces between the clay minerals and the water molecules must have a considerable influence upon the entropy of the system. This is seen from the rate of cooling of clays immerged in liquid air or carbon-dioxide-toluene mixtures. In Fig. 6 the temperature-time relations for cooling of moist clays, dry clay powder and pore water pressed out of the same clays are shown. It is conspicuous that when a moist clay is immersed in carbon-dioxide-toluene at a temperature of -78° C, we have several irregularities in the cooling curve, proving that exothermic reactions (probably freezing) are taking place. When the clay is immerged in liquid air at a temperature of -185° C, however, we have very smooth curves. Thus, it seems as if the exothermic reactions which were conspicuous in water and easily seen in the clays at the higher temperature of -78° C are absent at this low temperature. The possible explanation for this striking feature is that the "clay water" at the low temperature of -185° C is kept in a metastable state of higher energy than ice at the same temperature. We may compare this state with that of glass. At normal temperature, normal glass does not crystallize in hundreds of years. But if it is heated to somewhat above the weakening point, a fairly rapid crystallization takes place and the heat of crystallization is liberated corresponding to the exothermic reaction in clays at -78° C. By calorimetric measurements we find a marked difference between a clay cooled quickly and the same clay cooled slowly to -78° C. Further there is a very marked difference between the heat content

of such clays and the same amount of dry clay powder and water kept separately. For instance with a certain clay from Southern Norway containing 57,05% water we have found the following caloric values:

Clay W = 57,05% between the temperatures -78° C and +22° C.

1 gram of dry clay	16,87 calories
heat of wetting correction	1,1 calories
Sum, approximately	18 calories

0,5705 gram water (ice)	78,9 calories	(corrected for salinity	75,2)
1 gram clay	18.0 calories	Clay	18,0
Sum	96.9 calories		93,2

1,570 gram clay slowly cooled	88,4 calories
1,570 gram clay quickly cooled	82,0 calories

As seen from these figures, the heat content of 1,57 grams of clay quickly cooled is 6,4 calories higher than when slowly cooled; 11,2 calories higher than if the pore water phase and the minerals are separated; and 14,8 calories

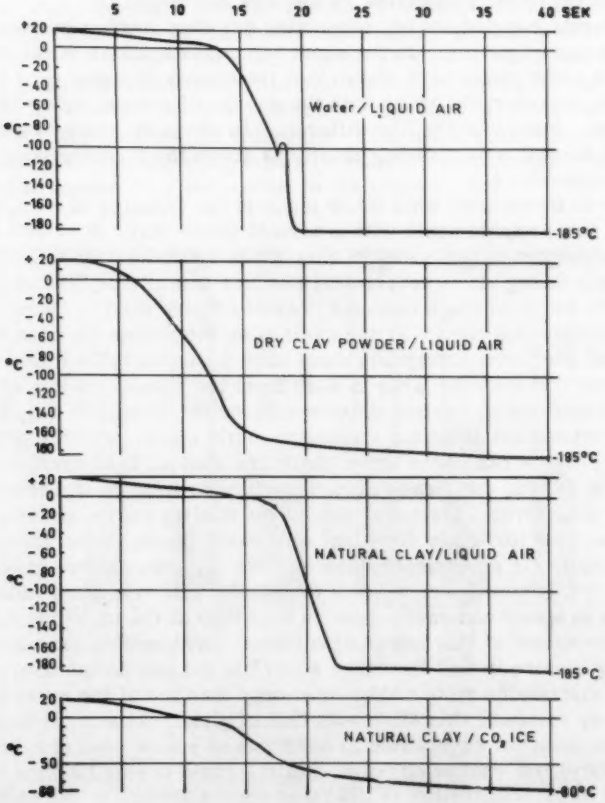


Fig. 6. Time-temperature curves for cooling of moist clays, dry clay powder, and pore water pressed out of the same clays.

higher than if the clay is separated into 3 parts, viz. minerals, dissolved electrolytes and distilled water. This clay had a salt content of 11,29 grams per litre of pore water. In more saline clays the difference between the two last figures will of course be higher, whereas in fresh water clays it will be insignificant. From these experiments we find that the free energy of the water in a clay is lower than the energy of free water at normal room temperature, and higher than the energy of ice at low temperature. Thus the freezing of a clay involves a much smaller change in the intrinsic energy of the water phase than the freezing of pure water.

Much importance has been attributed to the water phase nearest to the clay minerals. This water phase has been supposed by different authors to glue or weld the minerals together, thus being responsible for the cohesion term of clays. The concept of a relatively thick bound water layer around the clay minerals has entered into the clay literature at a very early time and is still often encountered. Measurements of heat of swelling and the vapor pressure/temperature relations of a particular clay substituted by different cations proved however that the calculated thickness of the "crystalline water" corresponds to an ice layer of 10A thickness surrounding the minerals. It seems, however, as if there is a gradual decrease in the degree of the order, so that the influence of the minerals will act upon the water phase to a somewhat greater distance, say 25A. To follow up this point, we have determined the vapor pressure/temperature curves for a clay with a known surface at a variable hydratisation over a range of water contents from practically dry to about the plastic limit. The values found confirm the exponential increase in free energy as a function of the distance from the mineral surface. Therefore, it is appropriate to speak about an adsorbed, diffused water film. Innermost, the water, at least, is influenced by the mineral in such a way that the bonds correspond to those in a solid material (ice). It is a well-known fact that even when dried at 110° C, the clays contain important amounts of hydrogen which will be given off by further heating. In case of illite clays, this hydrogen is supposed to be present in three different states:

(1) Hydrogen present as firmly adsorbed water (icelike water).
(2) Hydrogen present as H_3O^+.
(3) Hydrogen present as OH^- (lattice water).

The presence of the first group of water is inferred from the fact that there is but a gradual change in the slope of the dehydration curve when the heating is continued above 100°. The presence of hydrogen in the state (2) and (3) is directly seen form the structural formula of the illites.

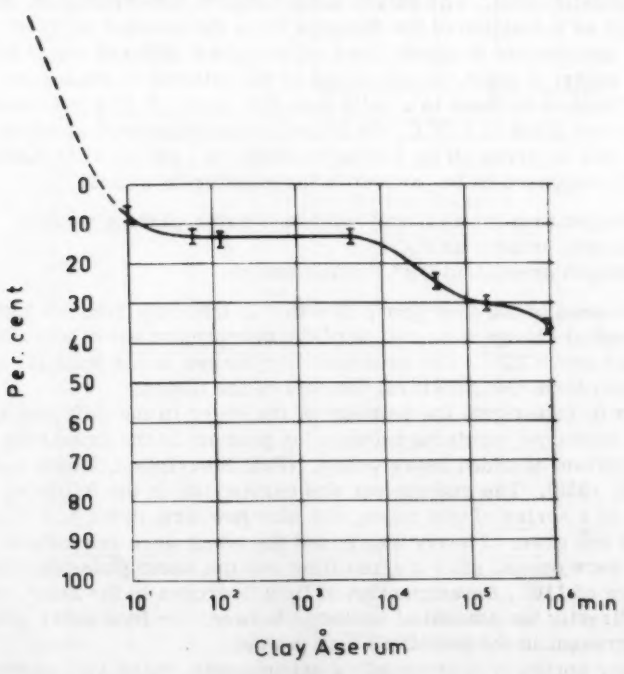
In order to investigate the mobility of the water in the different states, we have tried exchange reactions between the protium of the dried clay minerals and the deutrium of added heavy water. (Full description, Moum and Rosenqvist, 1958). The experiment was carried out in the following way:

In each of a series of test tubes, 10g clay powders, dried at 110°, were mixed with one gram of heavy water, and the tubes were immediately sealed. The tubes were opened after a given time and the water phase distilled at a temperature of 110°. Determination of light hydrogen in the heavy water would then give directly the amount of exchange between the free water phase and the water present in the pre-dried clay powder.

In another series of corresponding experiments, moist clay powder was mixed with a surplus of heavy water and immediately afterwards separated again by means of vacuum filters. This procedure could be done in a time

shorter than one minute. It proved, however, that even such a short time was not sufficiently short to give any idea as to the slope of the first exchange between the heavy water and an amount of water corresponding to that given off by the minerals by heating them at a temperature of 110°. Conventionally we call the dried residue of a soil heated to 110° C "the mineral phase" and the water expelled up to this temperature is called "the water phase;" this in spite of the fact that the liquid phase filling the voids in soils contains non-volatile dissolved substance, often in considerable amounts. Further, the soil dried at 110° contains considerable amounts of hydrogen in one state or other. The result of the different exchange reactions mentioned, was that the protium of the "mineral phase" underwent exchange reactions with the deutrium of the water phase at different velocities. One part of the hydrogen corresponding to about 13% of the total hydrogen content after heating to 110° C was completely exchanged with the surrounding water phase within a few minutes; another part corresponding to about 17% of the hydrogen content at 110° C needed weeks before equilibrium with the surrounding water phase was reached, and the third part corresponding to about 70% of the hydrogen content had not reached equilibrium even after 610 days; most probably several years—or even centuries--are necessary before 100% equilibrium is reached (Fig. 7). Eventually, however, all hydrogen atoms of the clay minerals will interchange and be in equilibrium with the free pore water.

Thus, we have to conclude that the normal clays will have their water present in different states. Some water fills up the voids and may be pressed



Exhonge of H/D in per cent of H content 110°C

Fig. 7

off mechanically by moderate forces. Conventionally we call this water the "pore water." This includes the so-called "capillary water." There is no physical basis for a distinction between "free pore water" and "capillary water," both terms describe mobile H_2O molecules in normal associations. The only difference is found in the hydrostatic pressure. Another part of the water cannot be pressed out by stresses normally applied, but will be expelled by moderate heating or drying in an atmosphere of low relative humidity. In addition to these two states of water, which represent more or less normal water, we have the three states of hydrogen mentioned above. It is, of course, a matter of convention where to draw the border lines between the different states. We prefer to make a separate group of the water content which is given off or taken up between room temperature at 96% humidity and 110^0 C at 0,6% humidity. This value is used because 0,6% relative humidity roughly corresponds to the water content of normal laboratory air when heated to 110^0. The water given off between these two limits we call the "hygroscopic" or "loosely adsorbed" water. As seen from the isotope exchange, there will, however, still be some water which does not differ qualitatively from this hygroscopic water, except by the degree to which it is adsorbed. We call it the "firmly adsorbed water". The rest of the hydrogen is then present as H_3O^+ and OH^- ions in the silicate lattice. These last states of hydrogen are generally accepted to belong to the mineral structure proper. All these different states of water are necessary in order to make up the natural mineral/water systems called clays.

The first water mentioned represents the pore liquid filling up the voids. This water is not considered as a part of the mineral phase, although it may be influenced by forces from the mineral lattices. The loosely and firmly adsorbed water may or may not be considered a part of the mineral phase. Study of the plasticity and shear strength of clay suggests, however, that it is natural to include at least the "firmly adsorbed" water in the mineral concept, as such an inclusion gives support to some reasonable hypotheses concerning the mechanical properties of the clay water system.

It has generally been assumed that the viscosity of the liquid phase in clays differs from the viscosity of normal water. Further, it has been assumed that the viscosity of the clay water increases with decreasing distance from the mineral surface. At present we do not know any technical method which permits a direct determination of a yield value for the viscosity of the water adsorbed on the surface of the tiny clay minerals. However, it is possible to make an indirect determination; because under certain conditions there exists a relation between the coefficient of the self-diffusion in a liquid and the coefficient of viscosity of the same liquid. (B. Ottar, 1955).

Provided diffusion takes place as single molecules, self-diffusion corresponds to average viscosities from 24 centiposes at 30% water and 153 centiposes at 10%. By extrapolating these average values, we arrive at very high viscosity values in the water phase next to the mineral surface, whereas the normal viscosity coefficient at about 1 centipose may be present in large portions of the free pore water (Fig. 8). This concept of a pitchlike viscosity in the water next to the minerals may give support to the opinion that the adsorbed water forms the "glue" responsible for the cohesion in clays. This primitive hypothesis, however, does not explain why the adsorbed cations have such an important influence upon the strength of a clay at a given water content. The difficulty arose as it was found that the viscosity and amount of

Relation between calculated average viscosity and water content in remoulded Aserum clay.

Fig. 8

adsorbed water seemed to vary but very little, whereas the shear strength varied considerably when the adsorbed cations were changed.

Another explanation as to the adhesion between the minerals in a clay may be raised in this connection, viz. the influence of variation in the surface energy of the adsorbed water and the water in the voids. It is generally known that the surface energy of solids is higher than that of liquids. As the free energy of the water next to the clay minerals is found to be nearer to that of ice than that of water, we have to assume that the surface energy of the adsorbed water might be higher than that of the free pore water. Thus, we can consider a condition similar to an "interphacial tension" between the water closest to the minerals and the free water of the voids. This "interphacial tension" may be suggested as a reason for adhesion between adjacent minerals and hence for cohesion even in saturated soils.

However, there is one observation which contradicts such an assumption, that is the fact that a partially dried clay with air in the pores has very much the same strength as when the pores are filled with CCl_4, although the interphacial tension of water towards CCl_4 is less than 2/3 of the surface tension of water.

The Effect of Water upon Mechanical Properties

Although the mineralogical composition, the grain size distribution, and the geological age and origin may differ radically from one clay to another, there are some general features which characterize all clays and separate them from other inorganic soils. The plastic properties of clay-water systems are the most conspicuous and most of the other characteristic properties of clays may probably be linked together with the fundamental origin of the plasticity. From Hvorslev's experiments we know that if a clay-water slurry is consolidated by a given stress to a given water content, and again unloaded and allowed to swell in water, the shear strength of such a material depends upon the water content and the stress conditions. The reduction in shear strength by unloading is relatively moderate for all clays and may be insignificant, as in the case of bentonitic clays. The shear strength of all clays depends upon the previous stress conditions in quite a different manner than with silts and sands. The shear strength remaining after unloading a clay must be physically explained by any theory describing the clay-water systems.

As mentioned, V. M. Goldschmidt found that the typical clay properties, mainly the plasticity, depended upon interactions between water and certain plate-like minerals. He found that if clay minerals were mixed with non-polar liquids as for instance carbon-tetra-chloride, the mixture behaved as a pure frictional material. He could not find any adhesion under such circumstances; the shear strength was proportional to applied stress, and disappeared when the normal stress was unloaded. Further, he found that it was impossible to obtain mixtures of clay minerals and carbon-tetra-chloride which could be shaped plastically in the same way as mixtures with water or other polar liquids. These were important findings and help us to understand clay-water systems. Goldschmidt worked with pre-dried mineral powders and did not try an exchange of water phase in undisturbed or remoulded clay-water samples. Such exchange experiments have been performed by Michaels and Lin (1954). They found that the structure and the permeability in prepared mixtures of kaolinite with different liquids at a given void ratio depended upon

the nature of the fluid. Even when water was used as the moulding fluid and was successively replaced by other liquids, the permeability values differed for different fluids; viscosities and densities taken into account. In these series of experiments we may infer that the structure of the mineral skeleton was unaltered by the replacement of one liquid for another. They point to the important fact that a part of the water in clay-water systems should be considered rather a part of the soil particles than a part of the pore fluid.

The consequence of the important work of Michaels and Lin is that presence of water in a clay-water system (1) influences the mutual arrangement of the mineral grains and (2) change them from being rigid silicate sheets into a new type of particles with immobilized hydrogen and oxygen atoms in a more or less rigid arrangement. A "clay-water" particle differs from a dry clay mineral because of the thin adsorbed water envelope. The isotope exchange experiments carried out at the Norwegian Geotechnical Institute confirm this opinion, and we have many reasons to regard the minerals in a clay-water system as thin plates where the external part has a decreasing yield value and modulus of elasticity. In mixtures of clay minerals with non-polar liquids, or when the minerals are dried, the plates must be considered somewhat thinner, but with uniform yield values, even in their outermost parts. It seems obvious that systems of so different a character must behave differently on remoulding and consolidation and give different structures even at the same void ratio.

It is a well known fact that the macroscopic shear strength and stiffness of clay-water systems increase by drying. At the same time we will have noticeable decrease in volume. In order to investigate the influence of the volume decrease upon the shear strength, we have tried to freeze dry undisturbed clay samples. This is accomplished by immersing a sample in liquid air (movement of the liquid phase is negligible), and then the ice is sublimed off by placing the sample on a vacuum above a drying agent (phosphoreus pentoxide) at a temperature well below the freezing point. If a clay is treated in this way, we will notice several interesting features: first, that the increase in volume due to freezing is less than what should be expected from the difference in volume between the water content in the liquid state and in the ice state; secondly, that there is no change in volume during drying; thirdly, we find that the shear strength and modulus of elasticity is much less in the freeze dried clays than in the normally dried clays. In fact the freeze dried clays do not seem to differ very much from the wet clays in their strength values. However, they behave as very brittle materials with a negligible deformation at failure, and not as plastic materials.

The freeze dried clay behaves as a material very different from a sample of dry clay powder compacted to the same volume on the one hand and a normally dried clay on the other. In the first case, we have no tensile strength, in the second case, we have a much higher one. We may replace the air in the freeze dried clays by a non-polar liquid such as carbon-tetra-chloride. This may easily be done without any harm to the clay sample. The dried clay does not fall apart and the strength values remain very much the same. If the freeze dried clays are put in water, they will at once disintegrate in the same way as a normally dried clay. If a clay with the voids filled with carbon-tetra-chloride is brought in contact with water, we will have a similar disintegration and carbon-tetra-chloride is ejected from the clay surface in small droplets. This shows that the wetting of the minerals must involve a

considerable loss of energy, probably due to the formation of adsorbed water films around each mineral.

All the different findings in undisturbed and remoulded clays with a liquid phase being either alcohol or aqueous solutions or a non-polar liquid as carbon-tetra-chloride illustrate that the important characteristic of the liquid is its polarity. The tensile or crushing strength of clays do, however, not seem to be fundamentally different in the cases when the liquid is of a polar or a non-polar character, or when the liquid is entirely lacking as in the case of the freeze dried clays, which consist solely of solids and gases. It seems as if the friction theory of Bowden and co-workers may fit in with this system of observations.

Some objections may be raised against the experiments described, mainly because of the fact that the freezing process involved a slight expansion, which may tear the single minerals somewhat apart. However, we may replace the water of a clay by replacement reactions similar to what was done by Michaels and Lin. If a natural undisturbed clay is percolated by alcohol ending up at 100% concentration, all the water which normally is considered as pore water and most of the adsorbed water will be removed. The system thus obtained behaves as a plastic material even though the plasticity is less pronounced than in the water system. This is similar to what was found by V. M. Goldschmidt on remoulded mixtures obtained from dry clay powder and alcohol. The alcohol-soil has shear strength properties similar to aqueous soils. Subsequent to the percolation with alcohol, we may bring in a non-polar liquid as carbon-tetra-chloride and we may replace all the alcohol in this way. Even this system has a considerable tensile strength, but has now lost all the plastic behaviour and seems to be very similar to the system of minerals and carbon-tetra-chloride obtained by filling the freeze dried samples.

When any two minerals are brought close to each other, bonds of adhesion between the grains will arise, independent of whether water is present or not. (The influence of the water is indirect. At a given distance, attraction depends upon the dielectric constant of the medium between particles). If the two minerals which are brought in contact are of the normal rockforming type as quartz, elastic stresses will arise in the areas around the points of contact, and when the external stresses are released, these elastic forces will tend to separate the minerals again. If, on the other hand, the minerals in question belong to the clay mineral group, they will in an aqueous system be surrounded by a layer of adsorbed water. When such minerals are pressed together, the outermost layer of the mineral (the water film) will yield plastically and little or no elastic stresses will arise at the areas of contact. For this reason a subsequent release of the pressure will not bring the minerals totally apart, and they will even in the unloaded state stick together because of the adhesional forces. With dry minerals or minerals immersed in liquids which cannot be sufficiently adsorbed, deformations will involve elastic stresses in the areas of contact and the material will behave as a pure frictional system independent upon whether the minerals are quartz or montmorillonite. The adhesional forces in clay-water systems are mainly of a Van der Wal's nature, perhaps also of inonic and covalent character. They depend upon the mutual distance between the minerals and the surface cations adsorbed upon them.

A model analogous to the cohesional and frictional properties of clay could be obtained from steel balls and a fine cover of a ductile metal like indium. The thickness of this plating must be very thin, of the order of magnitude 10A, and evenly distributed. J. S. McFarlane and D. Tabor (Proc. Roy. Soc. A 202,

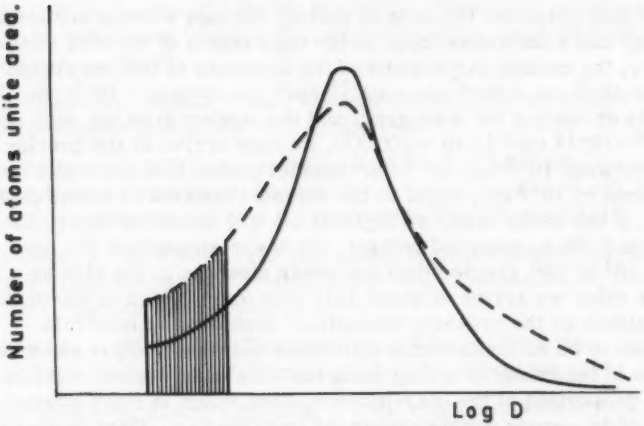
244, 1950) have shown that when a clean, hard metal ball is pressed into an indium surface, we will obtain a permanent adhesion and the tension necessary to break the bonds is of the same order as the normal stress which brought the surfaces together. In the model outlined above, the steel balls represent the clay minerals with the adsorbed ions and the adhesional forces should mainly be due to the atoms in the steel, whereas the idea of the indium film is to yield and prevent elastic stresses in the areas of contact.

The cohesion in a clay sample may be regarded as the integral adhesion. This part of the cohesion is therefore related to the non-dilatant friction and will depend upon the number of bonding atoms acting between adjacent minerals and their mutual distance. For a completely haphazard arrangement of the minerals, there will be one given distribution curve characterizing the distance between the atoms of adjacent grains. Under such circumstances we will have a given integral Van der Waal attraction and a given electrostatic ionic attraction and a given Coulomb repulsion between a single grain on one side of a supposed shear plane and the neighboring grains on the other side of this plane. Hence the adhesion, which is supposed to be the sum of all attractive and repulsive forces, will depend upon the mutual arrangement of the mineral grains as well as upon the nature of the grains.

If the water content of the clay-water system is changed, we will at the same time introduce a change in the distances between the grains and the integral attractive and repulsive forces will change. The cohesion term will hence depend upon the void ratio which with saturated soils means the water content. In this case it was supposed that the mineral grains continue to be distributed haphazardly, without any preferred orientation. We may on the other hand keep the void ratio (the water content) constant and alter the mutual orientation of the grains. The distance distribution function will then change accordingly. We know that it is the short distances, mainly the distances shorter than 15A, which are of importance. At greater distances the Van der Waal forces will decrease rapidly and even turn into repulsive forces. Therefore, it is obvious that the mutual orientation of the grains will have an important bearing upon the bonds between the single minerals. If we consider several samples having exactly the same minerals and water content, but each with a very much different orientation of the grains, the distance distribution function may be as illustrated in Fig. 9.

The stereoscopic pictures of sedimented clays (Fig. 4) show a certain degree of subparallelism in the horizontal plane, but mainly haphazard orientation, so that the characteristic contacts between a corner of one mineral and the plane of another predominate.

If we only look upon such a case, where all contacts are restricted to corners and planes, it is obvious that mathematically the contact area through a cross section will be equal to zero, as a corner is supposed to be zero-dimensional. Even when we have contacts between an edge and a plane, the contact area must be supposed to be zero, as the edges are supposed to be one-dimensional. Mathematically it will, therefore, not be possible to speak about any area of contact in such a system without giving the corners and the edges a certain area. As long as we are dealing with mass forces, it is obvious that we have to make such an assumption because the Van der Waal forces are acting over a certain distance and we, therefore, have to take a certain depth into account. Looking upon a corner of a mineral as a pyramid, it will be justified to speak about a "contact area" and set this area equal to a cross section through the pyramid at a distance from the point corresponding to the



Distance between atoms in neighbouring minerals.

SCHEMATIC ILLUSTRATION OF THE ATOMIC DISTANCE DISTRIBUTION IN A CLAY AT CONSTANT WATER CONTENT IN TWO DIFFERENT ORIENTATIONS OF THE MINERALS, i.e. BY TWO DIFFERENT STRESS HISTORIES. THE CROSSHEDGED AREA REPRESENT THE ATOMS INVOLVED IN THE ATTRACTION.

Fig. 9

effective average distances to the atoms which are active in the Van der Waal attraction between the pyramid and the plane. As the Van der Waal forces are decreasing at a high power of the distance, it will mainly be the atoms in the point of the pyramid which will cause the attraction, whereas atoms at a distance greater than 10-15 Å will only involve an insignificant attraction. For the sake of argument, we may therefore assume that the contact area between a corner and a plane will be of the order 10-100 $Å^2$ for each point of contact. When dealing with an edge and a plane, we have to take an effective plane through the wedge into account and the area would consequently be of the order 3-10 Å times the length of the contact. The shear strength of a sediment bonded solely by Van der Waal's forces and ionic bonds etc., but with no cementation will be proportional to the number of contacts per unit volume to the 2/3 power. If all minerals form 6 contacts, i.e., the number of contacts is 3 times the number of sides, this means that the shear strength of a clay paste should be proportional to the number of clay mineral grains per unit area. As the volume of the mineral phase in a unit volume = $\frac{100}{100 + G.w}$ and the volume of the median mineral = $(dv\ med)^3$, the number of contacts per unit area is $\frac{1}{(dvmed)^2} \times \left[\frac{1/2 \cdot 6 \cdot 100}{100 + G.w}\right]^{2/3}$. Here

dvmed is the median value of the volume diameter

G is the specific gravity of the clay mineral

w is the water content

The result thus obtained is approximately the same as can be calculated directly from a knowledge of the size and shape of the minerals. Under such

conditions we may calculate the area of contact through a cross section of 1 cm^2 in a clay and a thickness equal to the edge length of the clay minerals. Approximately, the median dimensions of the minerals in the Norwegian clays are 10^{-4} cm x 10^{-5} cm x 10^{-6} cm = 10^{-15} cm^3, i.e. dvmed - 10^{-5} cm. Supposing 6 points of contact for each grain and the contact area for each point between 10^{-15} -10^{-14} cm^2 (= 10 - 100 $Å^2$), we thus arrive at the conclusion that we have between 10^{-4} and 10^{-5} cm^2 contact planes in a slice of 1 cm^2 and the thickness of 10^{-5} cm, equal to the volume diameter of a clay mineral. Consequently, if the whole shear strength of the soil should be due to the adhesional forces in these points of contact, the shear strength in the contact area must be 10^4 to 10^5 greater than the shear strength in the clay as a bulk. As an average value we arrive at about 1-10 tons/cm^2, which is the same order of magnitude as the crushing strength of rockforming minerals.

There seems to be no fundamental difference between friction and cohesion. The difference in behaviour of a clay from that of a sand deposit must be due to the special properties of the clay-water system which in clays permit the areas of contact to remain nearly unchanged by unloading. What is normally called cohesion in soil mechanics means simply the shear strength intercept in a stress/shear strength diagram. Thus, a normally consolidated clay has a shear strength intercept equal to zero, whereas an overconsolidated clay may have a considerable shear strength intercept. It seems unreasonable that the cohesion should arise simply by unloading. As long as no cementation takes place, the shear strength intercept of an unloaded clay, according to the view presented, will depend upon the deformability of the aqueous part of the clay minerals.

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PHYSICO-CHEMICAL PROPERTIES OF SOILS: ROLE OF SOIL TECHNOLOGY

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FOREWORD

This symposium on the physico-chemical properties of soils was arranged under the auspices of the Committee on Mechanical Properties of Materials, of the Engineering Mechanics Division, sponsored jointly with the Soil Mechanics and Foundations Division.

Its objective was to summarize for civil engineers the current status of knowledge in this field and to stimulate further research and applications of this comparatively new and fundamental tool of soil technology.

The four papers in this symposium have been published as Proc. Papers 1998, 1999, 2000, and 2001.

SYNOPSIS

This paper first defines and describes the applied science called "soil technology." The powerful contributions that soil technology makes to soil engineering are next presented and illustrated. Some of the future developments in soil knowledge which can be expected are then discussed.

I. INTRODUCTION

The preceding papers in this Symposium on the "Physicochemical Properties of Soils" have presented material which is of considerable consequence to

Note: Discussion open until September 1, 1959. Separate discussions should be submitted for the individual papers in this symposium. To extend the closing date one month, a written request must be filed with the Executive Secretary, ASCE. Paper 2001 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 85, No. SM 2, April, 1959.

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soil engineering. The Author feels that the material will become increasingly important and that, therefore, this Symposium will prove to be a milestone in the activities of the Division of Soil Mechanics and Foundation Engineering.

This paper will attempt to point out the present and probable future significance to the soil engineer of the subject matter covered in this Symposium in the following manner:

1. List the areas of science which have been employed;
2. Suggest, describe and illustrate the contributions to soil engineering of these phases of science;
3. Indicate the limitations of the use of this material to engineering applications;
4. Suggest what future contributions the soil engineer may expect.

This paper will employ the term "soil technology" to define the applied science discussed in this Symposium. In particular, soil technology is the applied science which deals with the geological history and the composition of soil, and the influence of these two factors on the engineering behavior of soil. Soil technology considers not only the nature of the soil components, but also the arrangement of these components. It draws upon chemistry (especially crystal, colloid and inorganic), geology (especially mineralogy, sedimentalogy and physical geology, including glacial geology) and soil mechanics to explain soil behavior and how it may be altered. Soil technology thus includes the geological history of a soil along with the "physicochemical" properties of the soil.

At the outset of this paper it must be emphasized that soil technology is an integral component of soil engineering. The soil engineer hurts himself by considering soil technology as a separate area for the specialist only and progress in extending knowledge in soil technology is impeded if the problems of the soil engineer are not kept in mind.

In recent years there has been considerable advance in soil technology and many soil engineers are showing increasing interest in it. It is thus appropriate at this time to take stock of the role of soil technology in soil engineering and, in particular, attempt to show what help to the soil engineer soil technology is now and is likely to become.

II. Components of Soil Technology

The Symposium authors (Grim, Kerr, Taylor, Low, Rosenqvist, and Michaels) have clearly staked out the areas of basic science the soil engineer must study and, in fact, master in order to rise to, and maintain leadership in the profession. Below are listed these areas of science along with a few comments on how each aids the soil engineer.

A. Crystal Chemistry

Crystal chemistry treats the makeup of mineral crystals, and especially considers the forces between units of matter and the various atomic arrangements of crystals. Crystal chemistry is concerned with both primary valence bonds—those which link atoms in crystals—and secondary valance bonds—those which link crystals together.

The physical behavior of a mineral crystal depends on the nature and arrangement of the atoms which comprise it. The expanding character of

montmorillonite is, for example, explainable in terms of crystal chemistry. The nature of forces between soil mineral crystals is also most important to the soil technologist. Of particular importance are the magnitude of these forces and their variation with such boundary and environmental factors as temperature, pressure, water content, and pore fluid composition.

B. Colloidal Chemistry

Colloidal chemistry treats the behavior of particles that are of such small size and of such shape (large surface area) that their behavior is significantly influenced by electrical forces acting at the surfaces of the particles. Clay particles have a large surface area per mass and thus exhibit colloidal behavior. Two areas of colloidal chemistry are important to soil technology, namely: electrokinetic phenomena involving the movement of moisture relative to soil particles; and the interaction of neighboring soil colloids.

Colloid chemistry theory gives basic expressions for the relative movement of soil and water under four circumstances: a) electroosmosis—movement of water by an applied electrical current, b) electrophoresis—movement of soil by the application of an electric current, c) streaming potential—a potential generated by the movement of water, d) migration potential—the potential generated by the movement of soil.

Electroosmosis is important since it is a method of stabilization and also since it occurs during hydrodynamic flow because of the streaming potential generated by the flow. Electrophoresis is of limited importance—used to measure the electrical characteristics of soil particles. Migration potential is of no importance to soil technology.

Colloid chemistry treats, for certain ideal conditions, the effects of environmental factors on colloid stability. The Gouy-Chapman theory, for example, indicates the influence of temperature, dielectric constant, water content, ion valence, ion size and ion concentration on the stability of suspended soil colloids.

C. Inorganic Chemistry

The principles of inorganic chemistry are used in many phases of soil technology. Solubility and mass-action expressions are, for example, needed to study ion exchange reactions and such colloidal phenomena as osmotic pressure.

D. Mineralogy

The nature of the minerals in cohesionless soils is usually of little concern to the soil engineer. The minerals in fine grained soils, however, are usually of considerable interest. The type of minerals in clay determines ion exchange capacity, plasticity, water sensitivity, and has a major influence on strength, compressibility and permeability.

The structure of the minerals which occur as silts, sands, and gravels are reasonably well known. On the other hand, the structures of the minerals in clays are only approximately known. The last decade has seen considerable progress in working out the crystal structures of these minerals.

E. Sedimentology

Sedimentology covers the formation, transportation and deposition of sediments. The sedimentologist has made much headway in determining how

parent material, transporting agent and depositional environment influence the physical characteristics of individual sediments and the size and shape of sediment beds. Most of the developed knowledge is, however, qualitative and of somewhat limited direct practical value to the soil engineer.

F. Physical Geology

The value of geology to the soil engineer in giving information on the extent, shape and properties of soil deposits cannot be seriously questioned. While the soil engineering profession widely accepts the importance of geology, few practitioners really usefully employ geology in their work. The classic user of geology in soil problems is, of course, Terzaghi. He solves most of his consulting problems on the basis of an intimate knowledge of geology of the site involved. Terzaghi (1955) has clearly pointed out the contributions geology can make to soil engineering.

In addition to making this major primary contribution to soil engineering, geology feeds soil technology by giving information on post-depositional changes to soil deposits. Physicochemical soil data can seldom be properly interpreted without such geological information as: extent of uplifting, leaching, weathering, desiccation and precompression.

III. Contributions of Soil Technology to Soil Engineering

The contributions to soil engineering made by the principles of soil technology are, for the purposes of discussion, divided into three groups (the last two are essentially the same), namely:

1. Furnish data for direct use in a soil problem;
2. Help assign a value for a given soil property to be used in a soil problem; and
3. Help explain the fundamentals of soil behavior.

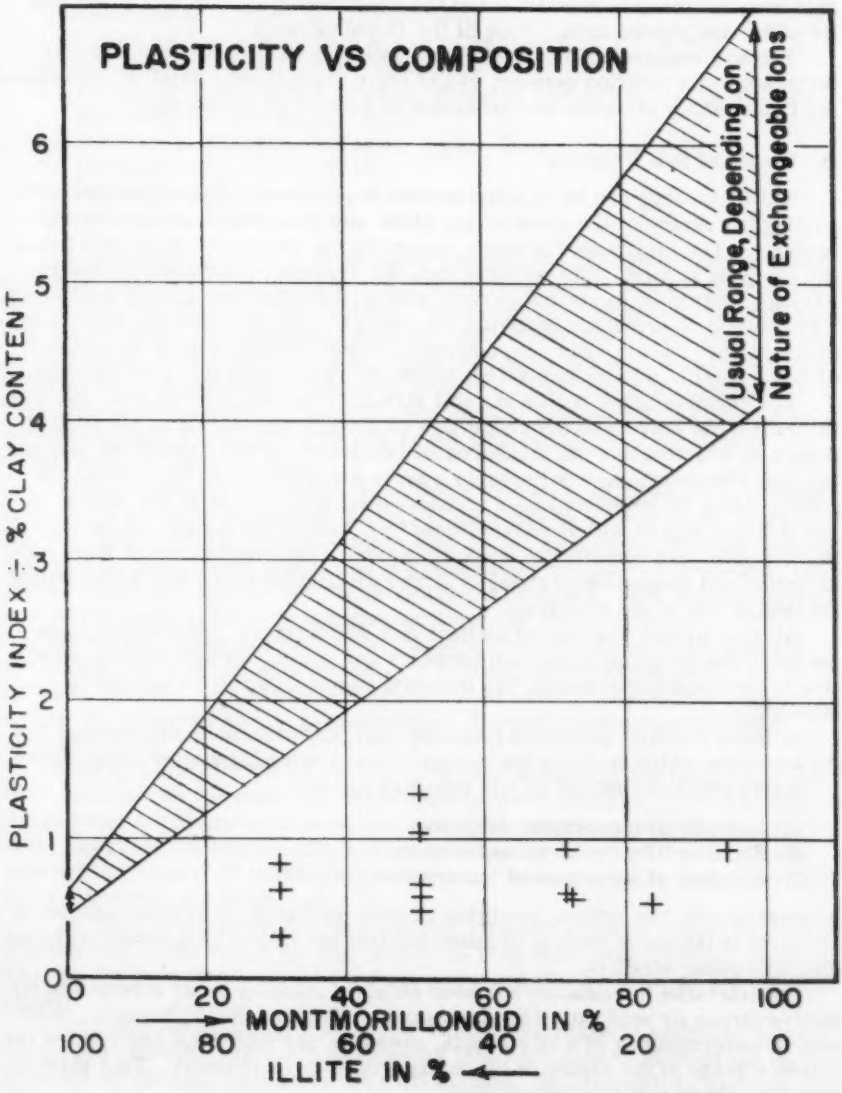
A. Furnish Design Data

The Author finds it difficult to cite many actual cases where numerical data obtained from compositional-type analyses of soil have been directly substituted in rational design formulas. Ion exchange capacity data have been used to compute the quantities of various additives to be employed in soil stabilization problems. Examples of this type are not numerous and are not of importance to the average soil engineer.

The basic soil properties do, of course, depend considerably on soil composition—minerals, exchangeable ions and "impurities." See White (1958) for an example of some data on the properties of homoionic clay minerals. To use data obtained on selected minerals in an attempt to predict the properties of a natural soil by apportioning the data from tests on the "standard" minerals according to the mineralogical make-up of the natural soil is nearly always doomed to failure, especially when predicting properties of an undisturbed soil. A natural clay very seldom behaves as a simple mixture of minerals acting independently of one another.

Fig. 1 (Lambe and Martin, 1957) clearly illustrates the type of failure to be expected if one tries to predict soil properties from compositional data on multi-mineral natural soils. In this figure, the plasticity index (divided by the clay content as determined by compositional analyses) is plotted against

PLASTICITY INDEX ÷ % CLAY CONTENT


PLASTICITY VS COMPOSITION
Usual Range, Depending on Nature of Exchangeable Ions
PLASTICITY INDEX ÷ % CLAY CONTENT
0
1
2
3
4
5
6
0 20 40 60 80 100
MONTMORILLONOID IN %
100 80 60 40 20 0
ILLITE IN %

the relative percentages of montmorillonoids and illites in a group of natural soils. If the principle of apportioning the properties according to composition held even on remoulded soils, all of the experimental points in Fig. 1 would lie within the shaded area. None of the 13 points does.

The soil engineer can correctly relegate this first contribution of soil technology to a position between no and little importance. Some of the reasons for this unhappy situation are presented in Section IV of this paper.

B. Choice of Soil Property

Soil technology can be of considerable importance to the soil engineer in guiding his choice of the value of any given soil property to be used in any problem. The importance depends, naturally, on the nature of the soil and of the problem at hand. The engineer can, for example, satisfactorily design footings resting on sands and gravels with no consideration of soil technology. The solution of problems involving the strength, compressibility, or permeability of fine grained soils can, however, be significantly helped by soil technology. Most soil problems arise, moreover, with these fine grained soils.

The engineer must design his soil structure for some given life. He must, therefore, not only know the properties of the soil in question at the time of design or construction (as measured by laboratory or field tests) but also determine ("approximate" is probably a more accurate word) the critical effective value the property under consideration will assume in the future. A fine grained soil is not necessary "dead" or "inert" but rather can be very much "alive" and sensitive to its environment. The engineer must know the direction and magnitude of changes in soil properties which will occur during the design life of the structure.

All clay properties depend on time and environment. Since strength is probably the most important soil property and since it is particularly sensitive to time and environment, the following discussion will be limited to strength.

In many stability problems involving clay, the critical condition—and thus the condition which controls the design—occurs some time after construction. Earth problems falling in this category include:

1. Stability of a pavement subgrade
2. Stability of a cut or unloaded slope
3. Stability of a compacted embankment.

In other words, the critical condition in these problems is reached, not by an increase in the loads tending to cause failure, but rather by a reduction in the effective shear strength.

Considerable information has been obtained relating shear strength to effective stress or void ratio (see for example, Taylor [1948]); however, to obtain an understanding of why strength, pressure and void ratio are related requires a study of the nature of the interparticle force systems. This study of "why" is where soil technology helps.

There are many factors, other than pressure, that influence soil strength and can change during the life of the soil structure under consideration. The principles of soil technology are essential to the soil engineer in his attempt to understand the effects of these other factors on soil strength. Soil technology has, for example, recently supplied qualitative information on the influence of these other factors on shear strength. For example, the principles of

colloidal chemistry indicate that, in general, a reduction in the shear strength of a clay will result from:

1. Reduction of electrolyte concentration
2. Cation exchange from high to low valence (e.g., Ca^{++} to Na^{+})
3. Adsorption of anions (e.g., phosphate)
4. Exchange from a cation of small hydrated radius to a cation of large hydrated radius (e.g., Na^{+} to Li^{+})
5. Increase of dielectric constant of pore fluid
6. Increase of pH of pore fluid
7. Decrease of temperature
8. Increase in water content.

The influence of some of these eight changes has been investigated experimentally. The Norwegians [Bjerrum, (1954); Rosenqvist, (1955); Bjerrum and Rosenqvist, (1957)] have, for example, conclusively shown that a reduction of electrolyte in the soil pores by leaching reduces both the undisturbed and remolded shear strengths of the soil. This knowledge not only helps in solving engineering problems involving these sensitive clays, but also suggests ways to reduce the sensitivity of the soil. Making salt available to the ground water percolating into a sensitive clay can result in a reduction of sensitivity and an increase in shear strength.

Taylor (1958) has also pointed out that a reduction of strength can result from the removal of polyvalent cations from a soil by ground water leaching. He even terms a clay which has more than 15 per cent of its exchange capacity satisfied by monovalent cations as in a potentially "dangerous condition" since ground water can remove the excess polyvalent cations. Such a leaching would permit the monovalent ions to cause swelling and strength loss.

The loss of shear strength from the adsorption of anions (Item 3 of the above list) has been studied extensively in connection with the action of dispersants on soil. The adsorption of polyvalent anionic groups is the major mechanism whereby the dispersants effect dramatic reductions in shear strength of soil.

Items 4, 5 and 6 of the above list have been studied in the laboratory but little is known on the practical importance of these changes. It is doubtful that Items 4 and 5 will prove to be of much practical importance to the soil engineer; however, Item 6, the effect of an increase in the pH of the pore fluid, may well be important particularly where soil reservoirs are being used to store industrial products.

The effect of temperature on the shear strength of clays should be of some consequence to the soil engineer. There is very little experimental evidence relating temperature to shear strength and this little evidence is not conclusive.

The reduction of strength resulting from an increase in moisture is well known and attempts are made to measure this reduction in most of the methods of pavement design, especially the CBR method. Soil technology helps in estimating the amount of water a given clay under a given set of boundary conditions will imbibe.

Time (along with pressure) can either decrease or increase the strength of a clay. In unloaded stiff fissured clays a dramatic loss of strength can occur as Terzaghi (1936) and Skempton (1948) have shown. On the other hand, a major increase in strength can occur because of "thixotropy" as the works of Moretto (1948), Skempton and Northey (1952) and Seed and Chan (1957) show.

Grim (1958) has indicated the detrimental effects which weathering can have. The leaching by ground water of such minerals as muscovite, biotite, chlorite, and illite tends to remove non-exchangeable cations from between the silicate layers. The removal of the potassium (from illite, muscovite and biotite) and magnesium (from the brucite layers of chlorite) tends to "degrade" the micaceous minerals i.e., tends to remove the "bridge" or "glue" which holds the silicate sheets together. The mineral particles thus subdivide, imbibe water and lose strength.

Grim emphasizes that alterations from weathering need not be a slow process measured in geological time, but can occur very rapidly, particularly under warm, moist, climatic conditions. He suggests that, under such favorable conditions of weathering, substantial changes can occur in a matter of a few years.

The foregoing discussion should indicate to the soil engineer how important the effects of time and environment can be on the shear strength of a fine grained soil. While probably not as important or as dramatic, changes with time and environment will occur in other clay properties such as compressibility and permeability. The profession can sincerely hope that the intensive study which is underway in many laboratories throughout the world will help to define more clearly the effect of time and environment on soil properties and especially give us more quantitative data on these changes.

In summary, soil technology aids the engineer considerably in his choice of property to use in a given problem. In many engineering problems involving clay, the area of uncertainty lies in the selection of the properties of soil to be used rather than in the actual analytical methods employed.

C. Explain Fundamentals of Soil Behavior

Soil technology offers the profession its main hope of explaining the fundamentals of soil behavior. This, the most important contribution of soil technology to soil engineering, is really just a broadening and extending of the second listed contribution discussed in the preceding pages. The pioneers in soil mechanics made sound and rapid advances in developing knowledge of soil behavior using conventional soil testing equipment and only little help from pure scientists. Laboratory research employing conventional techniques and not fully exploiting the principles of the basic sciences such as chemistry and physics would appear to have reached, or in some cases, passed the point of diminishing returns.

Someone has suggested that the consideration of soil mineral and pore water as a single inter-related phase rather than two isolated phases is the start of the "second revolution in soil mechanics," the first, of course, being that started by the unique work of Terzaghi. The science-flavored soil research of recent years has substantiated, broadened and put in more scientific useful terms some of the basic discoveries of such pioneers as Terzaghi and Hvorslev. For example, research (using techniques unavailable to Hvorslev during his studies) has proved correct Hvorslev's postulations on the role of particle orientation on clay compression.

Fundamental soil research has also discovered phenomena completely unknown; at least, the literature on the previous research made no mention of the phenomena. One example, of such a discovery comes from the work of Rosenqvist on clay leaching. He found the cause of the high sensitivity in the Norwegian quick clays.

In the coming years research employing the areas of science described in this Symposium will, in the Author's opinion, make additional contributions of both types—i.e., improve knowledge on know principles and discover new ones. History in other fields has shown unwise the tendency of some practioners to want fundamental research justified in terms of immediate practical results. The useful applications of many basic discoveries were not even considered prior to, or at the time of, the discoveries.

There is no need to emphasize to most engineers, especially to the researchers and the educators, the importance of a fundamental understanding of clay behavior. Since, however, the most important contribution of soil technology is to help explain the fundamentals of soil behavior, some of the areas of soil engineering where such fundamental knowledge is desirable or essential will be described.

1. Stability and Settlement Problems

A better understanding of the fundamentals of shear strength and compressibility of clays is a primary need for an improvement in the accuracy of the solution of soil stability and settlement problems. Plastic clays are being used in earth structures at an increasing rate. Colloidal effects which may be minor in lean clays can be of overriding importance in plastic, water-sensitive clays. A better understanding of water imbibition and its effects on the long term strength of a clay is, for example, needed for the proper design of embankments of clay. The importance of knowing the effects of time and environment on the strength of clay has already been emphasized in the preceding section. Knowledge of the causes of secondary compression would permit more intelligent estimates of settlements.

An excellent example of the important contribution soil technology can make to the explanation of soil behavior is furnished in the case of clay swelling. Recent work by Bolt and others [as described in Lambe (1958)] has clearly elucidated the mechanism by which saturated clays imbibe water and increase in volume. This knowledge aids the engineer in his attempts to predict soil expansion and, further, suggests practical means of reducing the expansion of a clay in contact with moisture (e.g., the addition of electrolyte to the clay pore fluid).

Soil technology has also helped in the study of shear strength. For example, soil technology can explain (on the basis of the electrical forces between particles) why certain soils lose all of their strength when the effective stress is reduced to zero and why others such as certain marine clays do not slake when submerged.

2. Soil Stabilization

Many of the techniques of soil improvement, i.e., soil stabilization, employ a soil-additive reaction. Any such reaction obviously depends on the nature of the potential reactants. Soil technology has considerably helped in developing new soil additives, e.g., dispersants (Lambe, 1954, 1956). Dispersants have been successfully used for low-cost soil stabilization. A reservoir in Maine was, for example, lined with a one-foot blanket of chemically treated soil which cost only 10% more than an equally thick blanket of untreated soil but had the sealing capacity of a ten-foot thick blanket of untreated soil.

The development of trace additives to increase the effectiveness of Portland cement as a soil stabilizer is another example of the success of applying the principles of chemistry to soil stabilization research. This

research has shown, for example, that the strength of certain soils stabilized with cement can be increased several 100% with 1% or less of sodium sulfate, a cheap chemical.

Soil technology can also help in the selection of stabilizer to be used with a particular soil. Examples: 1) High pH soils respond better to Portland cement than to asphalt; 2) carbonate soils respond well to cement treatment; 3) soils with soluble components respond to acid treatment for increasing permeability; and 4) soils carrying ferric or potassium ions are more difficult to disperse than those containing calcium.

The relationships between shear strength and environment as presented by the principles of soil technology suggest new and interesting techniques of stabilization. For example, on an earth dam containing clay the engineer could put a thin layer of polyvalent salt in the upstream portion of the dam. The water permeating through the dam would transport polyvalent cations dissolved from this salt into the dam. This leaching would result in an increase in shear strength of the clay in the central part of the dam.

Compaction is another subject where soil technology has contributed to soil engineering knowledge. Compaction of fine grained soils on the dry side of optimum moisture tends to give a flocculated particle arrangement, while compaction on the wet side tends to give a parallel arrangement. This difference in soil structure helps explain why samples compacted dry of optimum have higher strength and higher permeability than those compacted on the wet side. Seed (in a paper soon to be published by the ASCE) has extended these principles to explain why different types of compaction on samples of the same fine grained soil at the same moisture content result in quite different properties. This important work may lead to the point where engineers may someday specify, on the basis of laboratory tests, not only the amount of compaction and water content to be used, but also the type of compaction to be employed in the field. Continued research of this type might also lead to the development of cheaper and more effective methods of compaction.

3. Soil Testing

While soil technology seldom furnishes design data it can be essential in: the selection of the appropriate soil tests to be run; the right interpretation of the test results; and, the correct use of the results. This situation arises especially where the soil under study contains a pore fluid other than pure water or is composed of attapulgite, so called "expanding-lattice" minerals (the most important group are the montmorillonoids), or halloysite, or contains such aggregating materials as ferric oxides or carbonates. Several examples of this situation follow.

Theoretical considerations and experimental data have shown that the nature of the permeant can have important effects on the permeability of a fine grained soil; these effects are in addition to those readily explained by those permeant characteristics (temperature and viscosity) normally considered by the soil engineer. The permeability of a soil depends on other permeant characteristics—pH, dielectric properties and soluble salts.

Minor changes in soil permeability will occur by altering the double layer thicknesses of the soil particles with changes in the permeant characteristics. The most dramatic effects occur, however, from changes on the state of aggregation of the soil fines by variations in the permeant properties. A fine grained soil containing pore water high in salt can have its structure completely changed by leaching it with pure water. Thus a laboratory permeability test using distilled water can show a permeability far different from the

in situ value. Reducing the concentration of the electrolyte in the pore fluid causes the fines to disperse and move. These fines can collect at some point (e.g., bottom of permeameter) and result in a greatly reduced permeability, or migrate out of the permeameter and result in an increased permeability. Permeability testing on soils containing fines should include a consideration of the nature of pore fluid in the soil (as well as that likely to exist during the soil's life in the structure under consideration).

The proper testing of montmorillonitic soils can require an understanding of soil technology. The following example will illustrate this point. A standard consolidation test was run on a montmorillonitic soil; in this test (as is usual) the water made available for clay rebound was pure water. In this standard test, the sample rebounded to a higher void ratio than the value existing at the start of the test. As a matter of fact, the entire consolidation test was far from what was really desired.

Because of the great thirst of montmorillonite soils for water, the test sample probably started with high negative pore water pressures. The tester should have permitted the soil to reach an equilibrium condition similar to that existing in nature—i.e., start his test by letting the clay undergo some swell prior to loading. This initial swelling and the rebound upon unloading should have occurred with a liquid, similar in composition to the pore fluid, available for imbibition. The swelling and rebounding in pure water can be much greater than in salt water.

A classic example of the value of compositional data to a soils problem is furnished by the Sasumua Dam (Terzaghi, 1958). This 110 foot high dam in Kenya, Africa, was built essentially of a most unusual clay (with upstream and downstream rock fills). The extraordinary nature of the clay is illustrated by the following characteristics (Terzaghi, 1958):

	Liquid Limit	Plastic Limit
a. Natural, before drying	87%	54%
b. Dried at 105° C and powdered	58	39
c. As in (b) but treated with 0.4% tetrasodium pyrophosphate	47	37
d. Dried, powdered, and rehydrated 1 month	63	39

Results from a laboratory compaction test using approximately the Standard Proctor effort gave: max. dry density = 71 lb./cu. ft. and opt. molding water content = 47%.

Knowing that the apparently undesirable properties could be explained by the composition of the clay (halloysite) and did not necessarily mean the clay had undesirable strength, compression and permeability properties, Terzaghi carefully evaluated the clay and then recommended its use in the dam. The performance of the dam (Terzaghi, 1958 and Dixon et al, 1958) justifies this recommendation. The study of halloysite as a construction material for an earth dam disclosed several other successful dams built of halloysite.

4. Frost Problems

The design of stable foundations and pavements in frost areas on materials other than rock or clean granular soil is fraught with uncertainty. The principles of water migration under thermogradients and of ice lens growth

are only partially determined. Delineation of these principles could result in: 1) better criteria for determining frost susceptible soils, 2) better design techniques to prevent or overcome frost effects, 3) additives to render certain frost susceptible soils non-frost susceptible and, 4) the tools to enable the engineer to compute the amount of heave to be expected for a given soil under a given set of environmental conditions.

Employing the principles of physics, chemistry, thermodynamics, etc., Chalmers (1956), Gold (1957), Penner (1957), Martin (1958) and Jumikis (1956) have made significant progress in extending the classic work of Beskow (1935) and Taber (1930).

5. New Problems

Theoretical knowledge of soil behavior is particularly needed in those problems where a large mass of field experience has not been accumulated. Soil technology helps in problems with very plastic clays not only because such soils are very sensitive to boundary and environmental factors, but also because the performance data of these soils in structures are limited. The literature in soil engineering has, for example, little information on the behavior of plastic clays in earth dams. The engineer must lean heavily on theory, therefore, when employing a very plastic clay in a dam.

As the soil engineer of the future is required to make use of soils which have largely been avoided in the past, to design new types of structures, and to design for new environments, he will have to rely more heavily on theoretical concepts of soil behavior.

IV. Limitations to the Contributions of Soil Technology

There are, of course, reasons for the limited quantitative help of soil technology to the soil engineer. These reasons, listed below, are such that the main contribution of soil technology will continue as it is now—furnish an understanding of fundamentals of soil behavior.

A. Unknown Geological History

Most soil problems do not warrant the time and effort required to work out the geological history of the site involved. Even on those major projects which can afford such geological study, the geology can not always be delineated with the desired certainty or in the desired detail. One classic example emphasizes the latter limitation. The nature of the depositional environment —salt, or brackish, or fresh water—of Boston blue clay, one of the most studied clays in all the world, has yet to be definitely established.

B. Unknowns in Underlying Sciences

Those sciences which underlie soil technology contain many areas of uncertainty. The crystal structures of most of the minerals in the clays are yet to be definitely worked out. Of more importance to the soil engineer is the lack of a clear understanding of the electrical forces between clay particles. Soil technology will, of course, advance as such questions are cleared up.

C. Complexity of Soil

Soils are discouragingly complex. Seldom do clays consist of one mineral, but rather of two or more minerals, often interlayered. The soil particles

are not of one size and shape so that the formulas of colloidal theory can be used to describe quantatively their behavior; rather there is a tremendous variation in size and shape of particles in the clay. Very small amounts of such components as ferric oxide or carbonates can have a major influence on the behavior of the clay. The characteristic of a given mineral depends on such things as the degree of weathering. Clays are seldom homoionic.

The small size of clay particles only compounds the effects of heterogeneous composition of clay. The individual particles are, unfortunately, too small to be seen under a normal microscope.

D. Difficulty of Making Compositional Analyses

The complete determination of the composition of soil usually requires complex equipment, highly trained personnel and considerable effort. The complexity of soil composition, as noted in the preceding section, is the reason soil analyses are often so difficult to make and to interpret.

V. Probable Future Contributions of Soil Technology

The use of soil, especially fine-grained soil, as an engineering material may never be as exact as the use of such materials as concrete, metals, woods and plastics. The variability of soil from location to location and the dependence of soil properties on boundary and environmental factors will often prevent a soil analysis from being exact. There is not too much that can be done about the uncertainty in soil designs which arises from this great variability. (Improved exploration techniques will, of course, help in determining the extent of variability.) On the other hand, much of the uncertainty in soil problems, especially those involving fine grained soils, can be reduced by an improved knowledge of how soil behavior is influenced by boundary and environmental factors. It would thus appear that one of the most important areas of research in soil engineering is the study of the fundamentals of clay behavior.

While the profession has reason to expect significant progress in expanding the knowledge in soil technology, it should be patient and realistic in its expectations. Enough of the obstacles confronting the researcher have been described to indicate why future developments are going to be hard to make and are probably going to occur at a relatively slow rate. New analytical tools, techniques and help from a broadened and strengthened soil technology are the hope for a major breakthrough. A device and technique which would permit the individual particles in a clay to be seen and studied without disturbing the clay mass would, for example, be a powerful catalyst to fruitful soil research. Rosenqvist's technique looks promising as an answer to this problem.

To help pinpoint what will probably be the major future contribution of soil technology to soil engineering knowledge, the three related topics most needing clarification are next listed.

A. Clay Fabric

1. What are the arrangements of particles that occur in clays?
2. How do these arrangements change with time and pressure?
3. What is the nature of contact between clay particles?

4. What is a feasible technique to measure fabric directly?

B. Forces Between Clay Particles

1. What are the nature and magnitude of attractive and repulsive electrical forces between particles?
2. How can these forces be measured in a given soil?
3. How do these forces change with time, pressure and environmental changes?

C. Soil Moisture

1. What is the nature of moisture in clays?
2. How does the nature of this moisture change?
3. What are the pressures (positive and negative), that exist in soil moisture?
4. How can these pore pressures be reliably measured in clays—especially when they are below atmospheric pressure?

The extent of knowledge on these questions varies considerably; encouraging progress has recently been made on several of them.

Within the answers to these basic questions lie the answers to many important practical problems facing soil engineers. Answers to practical questions such as the following depend on answers to the above basic questions.

1. How much of the strength of a clay caused by precompression is permanent?
2. How can secondary compression be estimated?
3. What is the effect of ageing on the properties of a soil deposit?
4. What is a dependable strength of a clay when shear strains are very rapid? Or very slow?

One soil type—organic soil—of major concern to the engineer has not as yet received much attention from soil technologists. Almost nothing is known about the composition of organic soils and the influence of various types of organic matter on soil behavior. Major progress and understanding of the reasons for the behavior of organic soil is not expected in the immediate future.

VI. SUMMARY AND CONCLUSIONS

This paper has attempted to list the components of soil technology and describe the type of contribution of each component. Soil technology is an applied science which covers the physicochemical properties of a soil plus the geological history of the soil.

The two major contributions soil technology makes to the soil engineer are: 1) help in the estimation of the soil property to be used in any given engineering problem, 2) help explain the fundamentals of soil behavior. These are now very important contributions and, in the opinion of the Author, they will become of increasing significance to both the research and practicing soil engineer. The day may not be too far off when, on major engineering projects involving clay, the soil engineer will make compositional and structure analyses (including some estimate of the contact area between soil particles) prior

to the instigation of an experimental program involving the conventional engineering tests.

Steady and significant progress in soil technology will most likely be forthcoming. Use of fresh approaches, new techniques and equipment, and more help from science offer the researcher the greatest hope for a major breakthrough. His future efforts probably will and should be concentrated on the paramount problem facing the engineer—the accurate characterization of the strength, compressibility and permeability of a soil under any given set of boundary and environmental conditions.

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CONTENTS

DISCUSSION

Page

Physico-Chemical Properties of Soils: Clay Minerals, by Ralph E. Grim. (Proc. Paper 1998, April, 1959. Prior discussion: none. Discussion open until September 1, 1959.)
by Paul F. Kerr 73

Physico-Chemical Properties of Soils: Ion Exchange Phenomena, by A. W. Taylor. (Proc. Paper 1999, April, 1959. Prior discussion: none. Discussion open until September 1, 1959.)
by Philip F. Low 79

Physico-Chemical Properties of Soils: Soil Water Systems, by I. Th. Rosenquist. (Proc. Paper 2000, April, 1959. Prior discussion: none. Discussion open until September 1, 1959.)
by Alan S. Michaels 91

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PHYSICO-CHEMICAL PROPERTIES OF SOILS: CLAY MINERALS[a]

Discussion by Paul F. Kerr

PAUL F. KERR.[1]—Dr. Grim brings to this discussion conclusions reached after a long and able career devoted to clay mineralogy. There is little in principle that may be added to his presentation. However, a few items may be mentioned which Dr. Grim would doubtless have included if the limitations of time and space had permitted. Also, a brief independent approach to the same problem may be of benefit.

A number of years ago in the preparation of a glossary (Kerr and Hamilton, 1949) of the clay minerals it became apparent that a given clay mineral is included or omitted from a list of clay mineral species largely according to usage. About 15 minerals are ordinarily classed as clay minerals (Table 1). These belong to four groups: kaolin, montmorillonite, illite, and polygorskite. Fringe groups often closely related to the clay minerals particularly in occurrence would include the minerals of bauxite, the micas, the chlorite group, vermiculite, glauconite, alunite, talc, pyrophyllite, serpentine, sepiolite, and certain xenolites.

The micas in particular are closely related to clay minerals. Recently, through the work of the Geophysical Laboratory, the fundamental relationship of the micas and clay minerals has become better understood. In this work early appraisal suggested that a limited number of different types of micas probably existed on the basis of structural considerations. Laboratory synthesis (Yoder and Eugster, 1955) and structural identification (Smith and Yoder, 1956) followed. In the course of this work most of the micas have been synthesized and identified according to structural type. These are referred to as mica polymorphs (Table 2). Several forms of mica play an important role in sedimentary rocks. The micas are frequently found in nature accompanying the minerals of kaolin, illite, and montmorillonite.

Identification is nearly always a problem with clay minerals. Where independent clay mineral crystals exist, identification is more or less routine. Where two or more individuals may be present in the same aggregate, the separation and identification of the individual constituents is accomplished by conventional methods. In many instances, however, interlayer crystals are formed in which some combination of two different clay minerals may be present in single crystals. X-ray spectrometer study accompanied by lattice expansion utilizing previous treatment with glycol or other adsorptive media are usually applied to such crystals. The resolution of such interlayer units is often an important part of clay mineral identification.

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a. Proc. Paper 1998, April, 1959, by Ralph E. Grim.

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TABLE 1. Optical Properties of the Clay Minerals

Kaolin group:	Chemical composition	Crystal system	n_α	n_β	n_γ	$n_\gamma - n_\alpha$
Kaolinite	$Al_2O_3 \cdot 2SiO_2 \cdot 2H_2O$	Triclinic	1.561	1.565	1.566	0.005
(Anauxite)	$Al_2O_3 \cdot 3\pm SiO_2 \cdot 2H_2O$	Triclinic		(see Kaolinite)		
Dickite	$Al_2O_3 \cdot 2SiO_2 \cdot 2H_2O$	Monoclinic	1.560	1.562	1.566	0.006
Nacrite	$Al_2O_3 \cdot 2SiO_2 \cdot 2H_2O$	Monoclinic	1.557	1.562	1.563	0.006
Halloysite	$Al_2O_3 \cdot 2\pm SiO_2 \cdot 2\pm H_2O$	Aggregates		n=1.549-1.561		0.001
Hydrohalloysite	$Al_2O_3 \cdot 2\pm SiO_2 \cdot 4\pm H_2O$	Aggregates		n=1.526-1.542		
Allophane	$Al_2O_3 \cdot xSiO_2 \cdot nH_2O$	Amorphous		n=1.47 -1.49		
Montmorillonite group:						
Montmorillonite	$(Mg, Ca)O \cdot Al_2O_3 \cdot 5SiO_2 \cdot nH_2O$	Monoclinic	1.492		1.513	0.021
(Beidellite)	$Al_2O_3 \cdot 3SiO_2 \cdot nH_2O$	Monoclinic	1.517		1.549	0.032
Nontronite	$Fe_2O_3 \cdot 3SiO_2 \cdot nH_2O$	Monoclinic	1.580		1.615	0.035
Saponite Hectorite	$2MgO \cdot 3SiO_2 \cdot nH_2O$	Monoclinic	1.479-1.490	1.510-1.525	1.511-1.527	0.032-0.037
Illite group:						
Hydromuscovite	$KAl_2(OH)_2 [AlSi (O, OH)_{10}]$	Monoclinic	1.535-1.57		1.565-1.605	0.030-0.035
Palygorskite group:						
Palygorskite	$2MgO \cdot 3SiO_2 \cdot 4H_2O$ to $Al_2O_3 \cdot 5SiO_2 \cdot 6H_2O$	Monoclinic	1.510		1.533	0.023
Sepiolite	$2MgO \cdot 3SiO_2 \cdot nH_2O$	Monoclinic	1.490-1.506		1.505-1.526	0.015-0.020

TABLE 2. Mica Polymorphs

Structural nomenclature*	Rotation on basal plane	Space Group
1M	0°	$C_{2/m}$
20	180°	$Ccm2_1$
3T	120° or 240° (Mirror images)	$P3_112$ or $P3_212$
$2M_1$	120° or 240° (Alternating)	$C_{2/c}$
6H	60° or 300° (Mirror images)	$P6_122$ or P6522
$2M_2$	60° or 300°	$C_{2/c}$

*M, O, T, and H represent resepctively monoclinic, orthorhombic, trigonal, and hexagonal.

About ten years ago, a project was undertaken under the auspices of the American Petroleum Institute which carried the title: Research Project 49, Reference Clay Minerals.(1) At that time a number of research laboratories associated with the petroleum industry were initiating clay mineral research. Through the cooperation of a group of research workers in a number of laboratories a variety of data were assembled on large representative samples of the more common clay minerals. X-ray, spectrographic, chemical, electron micrographic, optical, infrared, thermal, ion-exchange and other data were assembled for each sample. The results were published by the Institute, and specimens were distributed to laboratories that indicated an interest. A large amount of clay mineral research over the last decade has been based on the specimens of Project 49, both in connection with the original studies of the project and independent investigations initiated since in other laboratories. Although all of the copies of two editions describing the work have been exhausted, the report of the project is available in most libraries, and microfilm copies are obtainable. Original specimens are no longer available for distribution, although a partial duplicate collection was made by a mineral dealer. Several laboratories still retain portions of the original specimens. The data of this investigation may be worthy of engineering reference.

The literature of petroleum geology also contains a number of studies of clay-bearing sediments which may prove of interest to some phases of engineering. The delta of the Mississippi River has long been one of the greatest natural clay depositories in the world. The total depth of deposition is unknown, but drilling indicates a section in which clay is abundant to a depth of at least four miles.

Thousands of specimens of these materials have been studied in the laboratories of oil companies and universities (Weaver, 1958; Adler et al., 1950).

Most of the applications of such studies have been along lines which pertain to petroleum geology. It seems likely, however, that a reappraisal of this work with a view to engineering problems would yield useful information.

Additional comments on clay mineral origin appear relevant. It is well to remember that clay minerals most frequently represent end products of rock destruction. A granite may be altered by natural processes and a portion of the rock mass or even the entire rock may be changed to clay minerals. The same applies to other substantial rock types. However, once the clay minerals are formed, subsequent changes are ordinarily negligible. The original rock, the clay minerals produced, and the extent to which these minerals have developed are significant factors in determining the physical properties of a rock mass.

The literature of mineral deposits contains much data which may prove of value to the engineering profession if it can be isolated and reappraised in terms of engineering problems. Some of this is reviewed in the proceedings of the First National Clay Conference at Berkeley (Pask and Turner, 1955) and in the publications which have resulted from subsequent conferences. In the areas of metallic mineral deposits in the western United States at Butte, Montana (Sales and Meyer, 1948), Bingham, Utah (Stringham, 1953), Santa Rita, New Mexico (Kerr et al., 1950), and Tintic, Utah (Lovering et al., 1949), clay mineral masses are formed by alteration of igneous rocks. The alteration represents a penetrating feature which may greatly change the physical characteristics of large rock areas. Since similar alteration may be encountered in engineering excavation, portions of this literature may have engineering significance.

Several techniques have been developed in recent years which are applicable to identification and description of fine aggregates such as the clay minerals. Few investigators apply all of these techniques to each individual specimen. The choice of those methods which will prove most rewarding in terms of time and effort is an important aspect of clay mineral work. An enumeration of the most generally used methods should be of interest.

Staining methods (Mielenz, King and Schieltz, 1950) are among the most rapid and are also applicable in some instances to field study. However, such methods are largely indicative rather than definitive and ordinarily require further confirmation.

The polarizing microscope (Rogers and Kerr, 1942), even after generations of application, is still a most useful instrument. However, the criteria of the polarizing microscope also require further confirmation.

One of the most useful methods for laboratory clay mineral study is the X-ray diffraction (Kerr et al., 1950). Diffractometer curves and diffraction photographs are widely used in clay mineral study. These are often definitive.

Another instrument receiving increasing attention is the split beam infrared spectrometer (Adler et al., 1950). With improvements in the technique of specimen preparation, this instrument has shown extensive utility.

Differential thermal analysis constitutes a method which is reasonably rapid and is widely used (Kerr, Kulp and Hamilton, 1949). As a supplement to X-ray or infrared examination, this method is particularly useful.

The electron microscope is employed on occasion to investigate the outlines of individual clay mineral crystals (Bates, 1958). These are only occasionally large enough to be distinguished as individuals with the polarizing microscope. The ordinary microscope, however, is particularly useful for the study of clay textures as revealed in thin sections.

Where chemical data are required, no substitute has yet been found for a reliable chemical analysis. Spectrographic data obtained with the emission spectrograph may also be found useful (Graf and Kerr, 1950).

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PHYSICO-CHEMICAL PROPERTIES OF SOILS: ION EXCHANGE PHENOMENA[a]

Discussion by Philip F. Low

PHILIP F. LOW.[1]—Ion exchange phenomena can be considered from the standpoint of thermodynamics, statistical mechanics, kinetic theory or double layer theory. Since Dr. Taylor based his excellent article on double layer theory, this discussion will be confined to that theory. In order to insure that the reader is acquainted with the basic concepts of the theory, a brief derivation will be given. Then the theory will be criticized.

The distribution of particles in a force field is given by the well-known Boltzmann equation. For any species of particle (ion, molecule, etc.), designated by the subscript i, the equation is

$$\frac{n_i}{n_{io}} = e^{(E_{io} - E_i)/kT} \qquad (1)$$

where n is the concentration of particles, E is their potential energy, k is the Boltzmann constant, T is the absolute temperature and the zero subscript designates a reference phase or state. When the equation is applied to the distribution of gas molecules (e.g. O_2) in the gravitational field of the earth, E_{io} is assigned a value of zero at the earth's surface. Then $E_{io} - E_i$ in the exponent of the equation becomes equal to $-m_i gh$, where m is the mass of the gas molecule, g is the acceleration of gravity and h is the height above the surface of the earth. Thus, the equation predicts that the concentration of gas molecules will decrease exponentially with height. Such is the case.

When the equation is applied to the distribution of cations in the electric field of a negatively charged clay particle, E_{io} is assigned a value of zero at an infinite distance from the particle. In this instance $E_{io} - E_i$ becomes equal to $-z_i\epsilon\psi$, where z is the ionic valence, ϵ is the electronic charge in e.s. units and ψ is the electrostatic potential. This equality is apparent if one recalls that ψ is the work done in bringing a positive e.s. unit of charge from infinity to the point in question and $z_i\epsilon$ is the charge on the ion in e.s. units. Since z_i is positive and ψ is negative, the cation concentration near the clay particle should exceed that in the so-called external solution, which is the solution at an infinite distance from the particle. These relationships are summarized in Fig. 1.

The effect of a space charge on the space rate of change of the electric potential gradient is given by Poisson's equation. To derive this equation in an elementary way, we will adopt two common conventions of electrostatic

a. Proc. Paper 1999, April, 1959, by A. W. Taylor.

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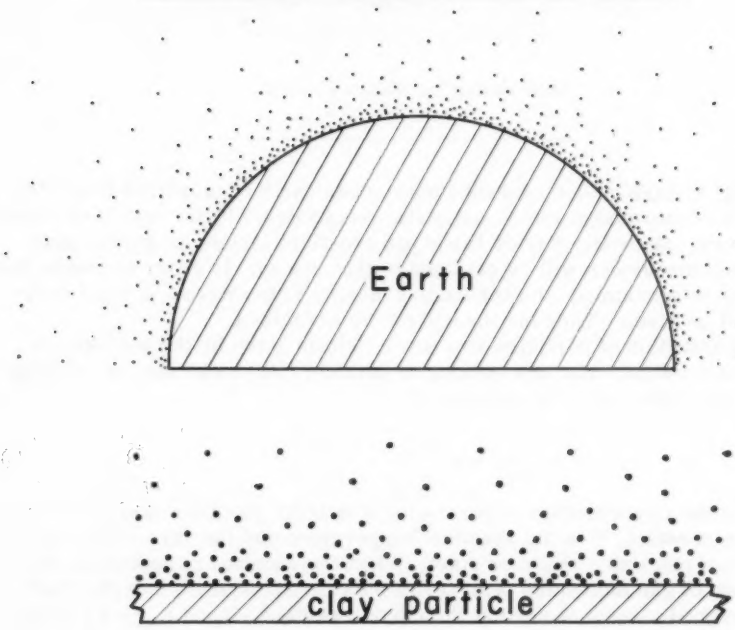
THE BOLTZMANN EQUATION



$$\frac{n}{n_o} = e^{\frac{E_o - E}{kT}}$$

n = concentration
E = potential energy
k = Boltzmann constant
T = temperature

GRAVITATIONAL FIELD

$$E_o - E = -mgh$$

m = mass
g = acceleration of gravity
h = height

ELECTRICAL FIELD

$$E_o - E = -z \epsilon \Psi$$

z = ionic valence
ϵ = electronic charge
Ψ = electric potential

theory, namely, (1) 4π lines of force emanate from each positive e.s. unit of charge and (2) the electric field intensity (force acting on a positive e.s. unit of charge) is equal to the number of lines of force per sq. cm. in a vacuum. Now, consider an infinitesimal, rectangular parallelepiped placed at right angles to the electric field of a condenser plate (Fig. 2). Let the distance between opposite faces in the direction of the field be dx and let the area of these faces be unity. If the parallelepiped contains positive point charges, more electric lines of force will leave the right hand face than will enter the left hand face, because 4π lines of force originate on each of the enclosed charges. Since the total charge inside the parallelepiped is equal to the product of ρ, the charge density, and dx, the volume of the parallelepiped, the difference between the number of lines of force entering and leaving the parallelepiped will be $4\pi\rho\ dx$. This difference will be equal to the change in field intensity between the two faces because their cross-sectional area is unity. Thus,

$$\frac{d\mathcal{E}}{dx}\ dx = 4\pi\rho\ dx$$

and

$$\frac{d\mathcal{E}}{dx} = 4\pi\rho$$

But the field intensity is equal to the negative gradient of the electric potential. As a result,

$$\frac{d\mathcal{E}}{dx} = -\frac{d}{dx}\left(\frac{d\psi}{dx}\right) = -\frac{d^2\psi}{dx^2}$$

Combining the last two equations, we have

$$\frac{d^2\psi}{dx^2} = -4\pi\rho \qquad (2)$$

which is the Poisson equation. If negative charges had been enclosed in the parallelepiped there would have been more lines of force entering the box than leaving it because lines of force terminate on negative charges, but the final result would have been the same. The equation's derivation is summarized in Fig. 2.

If the space charge in an electric field is made up of ions, the space charge density due to any ionic species is ϵzn since ϵz is the charge per ion and n is the ionic concentration. The total space charge density, ρ, is the sum of the space charge densities of the different ionic species. Therefore, it is given by the equation

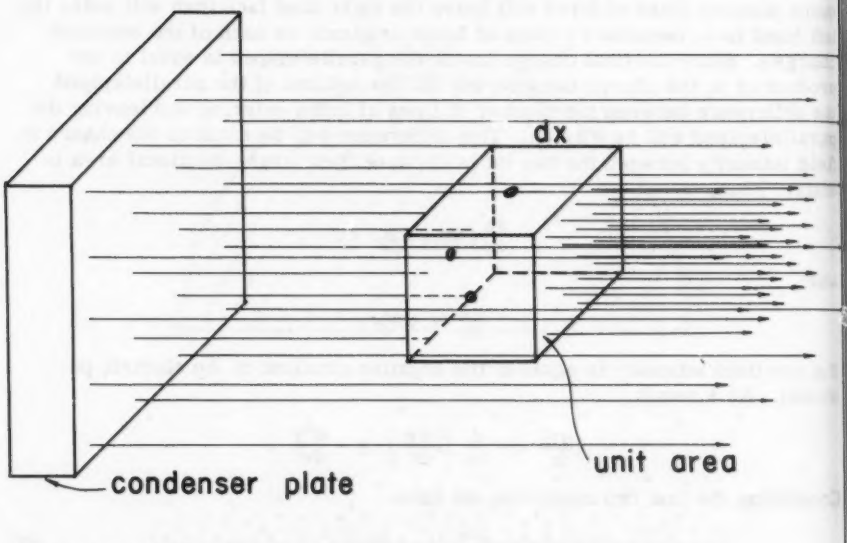
$$\rho = \epsilon \Sigma z_i n_i \qquad (3)$$

where Σ is the summation sign. Now, if the expression for n_i from Eq. (1), with the appropriate exponent, is substituted into Eq. (3) and the resulting expression is substituted for ρ in Eq. (2), we obtain the Poisson-Boltzmann equation

$$\frac{d^2\psi}{dx^2} = -4\pi\epsilon \Sigma z_i n_{io} e^{-z_i \epsilon \psi / kT} \qquad (4)$$

which is the basic differential equation referred to by Taylor. If the dielectric medium is not a vacuum, the right hand side of this equation is divided by D, the dielectric constant.

THE POISSON EQUATION



Conventions:

(1) 4π lines of force emanate from each positive point charge

(2) field intensity = number of lines of force per sq. cm. in vacuum.

$$\frac{d\mathcal{E}}{dx}dx = 4\pi\rho.dx$$

$$-\frac{d\mathcal{E}}{dx} = \frac{d}{dx}\left(\frac{d\Psi}{dx}\right) = \frac{d^2\Psi}{dx^2} = -4\pi\rho$$

$\mathcal{E}$ = field intensity ρ = charge density

x = distance

The flat, negatively-charged clay particle with its loosely-held, exchangeable cations can be regarded as a negative condenser plate surrounded by a positive space charge. Consequently, for a single clay particle we can integrate Eq. (4) between limits corresponding to infinity and the particle surface to obtain curves of the type shown in Fig. 2 of Taylor's paper. Or, for two parallel clay particles we can integrate Eq. (4) between limits corresponding to the mid-plane between the particles and the particle surface to obtain curves of the type shown in Fig. 3 of Taylor's paper.

According to double layer theory, the swelling of clays is due to the difference in osmotic pressure between the mid-plane between adjacent particles and the external solution. To illustrate the concept, consider the prune-water system of Fig. 3. The prune is surrounded by a membrane which is permeable to water but not to sugar molecules. When the prune is placed in water, water passes through the membrane into the region of high sugar concentration and causes the prune to swell. We say that it swells by osmosis. The pressure required to prevent the prune from swelling is called the osmotic pressure.

In the clay-water system there is no membrane to confine the solute—the ions in this case. But the electric field of the clay particles serves the same purpose. It restricts ionic movement and causes cations to accumulate in the inter-particle region so that the ion concentration in this region exceeds that in the external solution. And since water always tends to move into regions of higher solute concentration, whether the solute be sugar or ions, water moves into the inter-particle region to force the particles apart. Therefore, the pressure required to keep the clay from swelling can be regarded as an osmotic pressure. For both prune and clay the osmotic pressure is given by van't Hoff's equation

$$P_m - P_o = kT\Sigma(n_{im} - n_{io}) \tag{5}$$

Here P denotes pressure, the subscript m denotes either the interior of the prune or the mid-plane between clay particles and the subscript o denotes the external solution. The other symbols retain the meaning assigned earlier. These concepts are summarized in Fig. 3.

When Eq. (5) is applied to the swelling of clays, we can replace n_{im} by its equivalent from the Boltzmann equation with the result that

$$P_m - P_o = n_{io}kT\Sigma(e^{-z_i \epsilon \psi_m / kT} - 1)$$

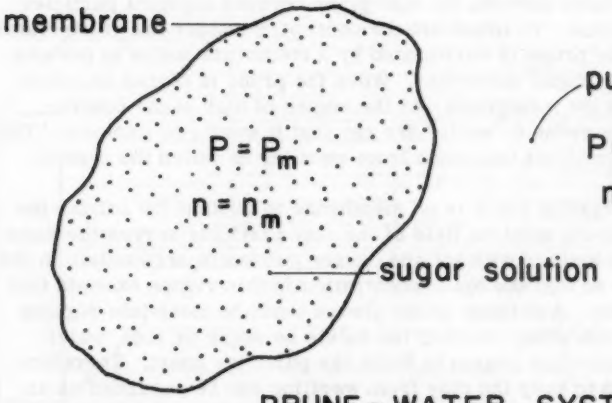
If only a single symmetrical electrolyte (z is the same but of opposite sign for cation and anion) is present, this equation can be converted readily to that given by Taylor or to

$$P_m - P_o = 2n_o kT(\cosh \frac{z\epsilon \psi_m}{kT} - 1)$$

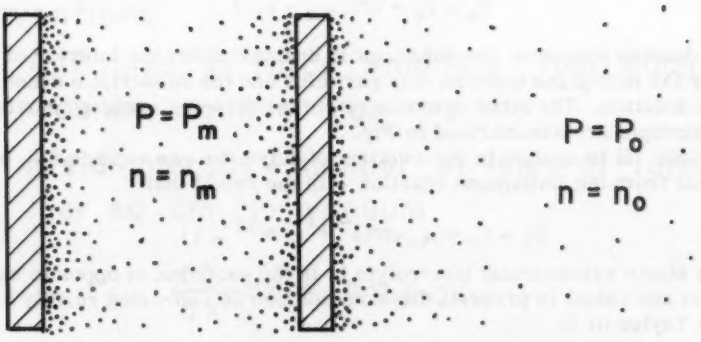
Since the value of ψ_m can be obtained from the integrated Poisson-Boltzmann equation, we can calculate the swelling pressure of the clay from these equations.

Now let us examine double layer theory critically. One assumption of the theory is that E_i in the Boltzmann equation is equal to $z_i \epsilon \psi$, the potential energy of the ion in the electric field of the clay particle. However, there might be additional components of E_i such as the polarization energy of the ion in the electric field of the clay particle, the energy of interaction between

SWELLING



PRUNE - WATER SYSTEM



CLAY - WATER SYSTEM

Van't Hoff's law:

$$P_m - P_o = kT \sum (n_m - n_o)$$

the ion and the surrounding ions or water molecules and the energy of interaction between the ion and the surface atoms of the particle. Bolt[3] made a theoretical examination of these additional components and arrived at the conclusion that their net effect should be small, excepting that they might influence the ratio of different ions adsorbed in the double layer. Let us study their net effect in a different way. We will consider an aqueous clay suspension containing two monovalent ionic species, A^+ and B^+. The distribution of each species between the particle surface and the bulk of the solution will be given by the Boltzmann equation. If we multiply this equation for species A^+ by the inverted equation for species B^+, we obtain

$$\frac{n_A}{n_{AO}} \cdot \frac{n_{BO}}{n_B} = \exp (E_B - E_A) + (E_{AO} - E_{BO})/kT$$

But, from thermodynamic theory we have

$$\frac{n_A}{n_{AO}} \cdot \frac{n_{BO}}{n_B} = K$$

where K is the equilibrium constant for the reaction

$$A^+ + B - \text{clay} \rightleftharpoons B^+ + A - \text{clay}$$

Thus,

$$\exp (E_B - E_A) + (E_{AO} - E_{BO})/kT = K$$

If the usual assumptions of double layer theory are valid (E_B and E_A equal $z_B\epsilon\psi$ and $z_A\epsilon\psi$, respectively, and E_{AO} and E_{BO} equal zero) the value of K should be unity. However, experimental evidence[8,9] indicates that the value of K, for monovalent-monovalent ion exchange on different clays, generally varies by a factor of about 20 but can vary by a factor as large as 350. Consequently, it is incorrect to assume that only the electric field of the clay particle affects the potential energy of the ions.[2] Other force fields, interionic or otherwise, must also exert their effects.

When double layer theory is applied to the swelling and compressibility of clays, the afore-mentioned assumption is invoked. In addition, the assumption is made that the clay surface does not attract water, i.e., it is lyophobic. If the clay surface attracts water so that the potential energy of the water is less at the median plane between adjacent clay surfaces than it is in the external solution, an additional term must be added to Taylor's swelling pressure equation. According to Low and Deming[10] this equation would become

$$P_s = C_o RT (C_m/C_o + C_o/C_m - 2) + (\theta_{wo} - \theta_{wm})/\bar{v}_w$$

where θ_{wo} and θ_{wm} are the potential energies of the water in the external solution and at the median plane, respectively, and $\bar{v}_w$ is the partial molar volume of the water. Now let us see whether or not the clay surface attracts water. If it does, then the contrary assumption is not valid and the usual swelling pressure equation is incomplete. Any alteration in the structure or properties of the adsorbed water that cannot be ascribed to the exchangeable cations will be regarded as an indication of clay-water attraction.

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2. Note that the incorrect assumption requires that the ions act as point charges in an ideal solution.

Recently, Anderson and Low[1] found that the specific volume of water adsorbed on montmorillonite surfaces was greater, i.e., the density was less, than that of normal water as far out as the median plane between adjacent surfaces at maximal swelling. The reduced density of the adsorbed water indicates that it has a more coherent structure than normal water.[3] And since ions would tend to break down any water structure and increase its density, the adsorbed water structure must be attributed to attractive forces at the surface of the clay. These surface forces could be either short-range forces transmitted by successive interaction of neighboring molecules to the observed distances, or they could be long-range forces acting directly on the water molecules. In either case, the potential energy of the water molecules would be progressively decreased with decreasing distance from the clay surface.

The coherent structure of the adsorbed water, if it exists, should impart a relatively high viscosity to this water. In keeping with this prediction, Low[11] found that the activation energy for ion movement[4] in the pores of a clay paste was greater than the activation energy for movement of the same ions in pure water. The activation energy increased as the specific volume of the adsorbed water increased, i.e., as the water structure became better developed. In addition, unpublished data, obtained recently by G. R. Dutt, show that the diffusion coefficients for LiCl, NaCl and KCl through clay pastes containing exchangeable Li, Na and K, respectively, are in the order expected on the basis of the experiments on adsorbed water density and activation energy for ion movement. The combined data from the three experiments, which were carried out on the same clays, are shown graphically in Fig. 4.

Additional evidence for a surface-induced increase in viscosity of the adsorbed water has been provided by Rosenquist.[15] He studied the diffusion of deuterium oxide (heavy water) through the pores of a clay paste and found that the diffusion coefficient was far less than would be expected if the pores were filled with normal water. Wang[16] has shown that, in normal water, dissolved ions often increase the diffusion coefficient of deuterium oxide by disrupting the water structure.

Evidence for clay-water attraction can also be obtained from freezing studies on clay pastes. In certain of these studies[4,7] the milliequivalents of exchangeable cations and the amount of unfrozen water at -5° C. were both determined. If the results of these determinations are used to calculate the maximum concentration of cations in the unfrozen water, it becomes apparent that the cations are not sufficiently concentrated to prevent the freezing of this water. And since swelling pressure measurements on the same clays[7] show that the pressure of the water is sufficient to lower its freezing point by only a fraction of a degree, the lack of freezing must be attributed to some other factor. Babcock and Overstreet[2] have presented an equation which shows that attractive forces at the clay surface can also lower the freezing point of water. It is postulated, therefore, that clay-water forces are the "other factor" that accounts for the observed amounts of unfrozen water.

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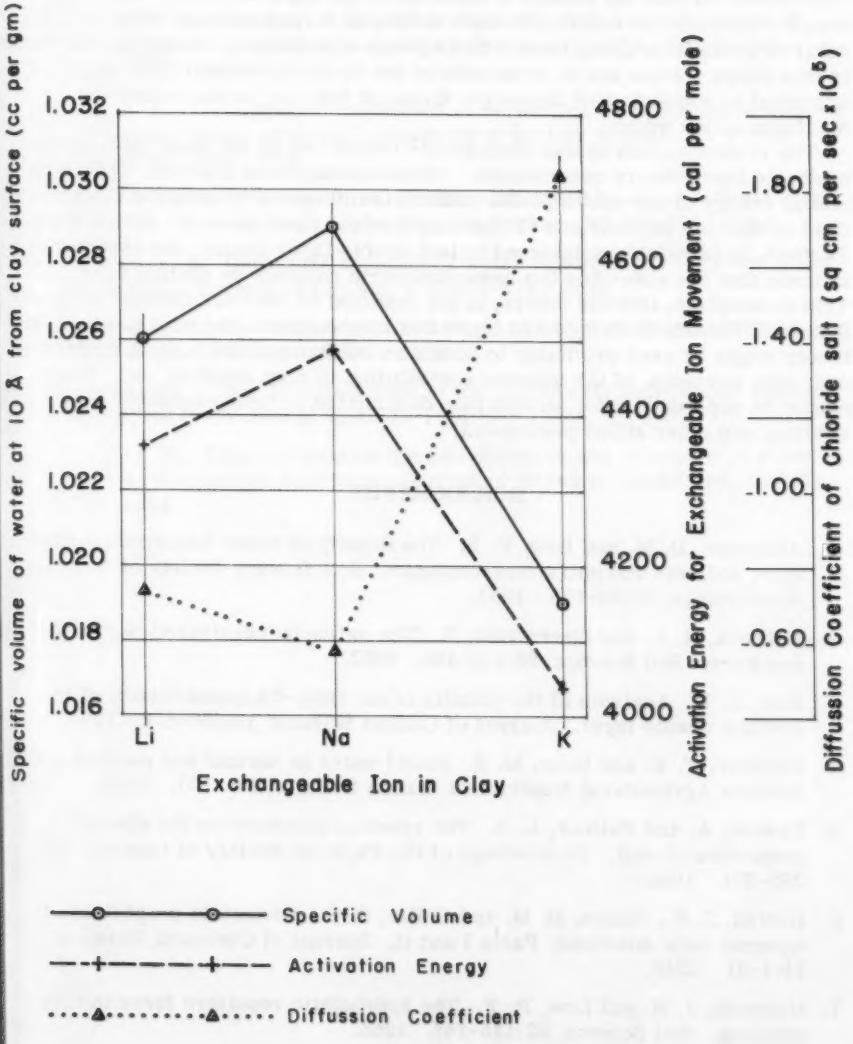
3. Water exhibits maximum structural development in ice with a density of 0.917. Water would exhibit minimum structural development in a close-packed arrangement with a density of 1.84.
4. In theory, the activation energy for ion movement is the sum of the energies expended by the ion in pushing back water molecules to form a hole and in breaking bonds with the neighboring water molecules and/or the clay so that it can move into the hole thus formed.


Specific volume of water at 10 Å from clay surface (cc per gm)
1.032
1.030
1.028
1.026
1.024
1.022
1.020
1.018
1.016
Li
Na
K
Exchangeable Ion in Clay
4800
4600
4400
4200
4000
Activation Energy for Exchangeable Ion Movement (cal per mole)
1.80
1.40
1.00
0.60
Diffusion Coefficient of Chloride salt (sq cm per sec x 10⁵)
Specific Volume
Activation Energy
Diffusion Coefficient

Dielectric constant measurements support the concept that attractive forces, which alter the water structure, exist at the surface of clay particles. Muir[12] and Palmer and colleagues[5,13,14] determined the dielectric constant of water in different clay systems at several moisture contents. They observed that the dielectric constant of the adsorbed water was very low; for example, on one clay sample it ranged from a value of about 3 to a value of about 50 in going from 0 to 89 per cent moisture. Dielectric constants in this range are too low to be accounted for by dissolved salts.[6] Again, it is logical to conclude that attractive forces at the clay surface alter the structure of the water.

The evidence cited in this discussion[5] makes two of the basic assumptions of double layer theory questionable. These assumptions are that: (1) the potential energy of the exchangeable cations is influenced by only the electric field of the clay particle and (2) the clay surface itself does not attract water. Further, in calculations designed to test double layer theory, the assumption is made that the water has the same dielectric constant as normal water. This assumption, like the others, is not justified by the experimental evidence. It seems, therefore, that double layer theory should be used with caution. The theory might be used profitably to obtain an estimate of the ionic distribution near clay surfaces, of the osmotic contribution to clay swelling, etc. But it cannot be depended on for an accurate description of ionic exchange, clay swelling and other allied phenomena.

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PHYSICO-CHEMICAL PROPERTIES OF SOILS: SOIL WATER SYSTEMS[a]

Discussion by Alan S. Michaels

ALAN S. MICHAELS.[1]—Dr. Rosenquist is to be highly commended for a splendid compilation of a large body of fact relating to the structure and properties of clay soils, and for a discriminating analysis of these facts in the light of modern colloidal and surface-chemical principles. His conclusion that the cohesive characteristics of clay soils are primarily attributable to coulombic (i.e., electrostatic), dipolar, and dispersion forces acting between adjacent particles in a soil mass is, in the opinion of this writer, incontrovertible in the face of the data now available. Perhaps the greatest single contribution of this work is the visual proof, via his ingenious electronphotomicrographic technique, of the existence of a "card-house" structure in undisturbed marine clays; this development definitely paves the way for a clearer understanding of cohesion in soils, and for more persuasive research into the origins of soil structure.

While Dr. Rosenquist's ultimate conclusions concerning clay-particle interactions and their relation to cohesion are in essential agreement with our own, there are (as might well be expected in as complex a problem as this) differences between our interpretations of the facts leading to our common conclusions. What I therefore wish to do is first, to discuss briefly the major points of interpretive disagreement, and second, to propose some alternative hypotheses which can also explain the experimental facts. Perhaps these counter-proposals, when compared with those of Dr. Rosenquist, may suggest avenues for further research which will even more satisfactorily clarify the soil-structure picture.

1. Exchangeable Cations, Swelling, and Liquid Limit of Clays

There appear to be inconsistencies in the relationship between the swelling in water, liquid limit, and the nature of the exchangeable cations in various clay minerals. With "expanding-lattice" clays of the montmorillinoid and (to a lesser degree) illite types, it is generally observed that, in contact with liquid water, the interlayer sorption of water <u>decreases</u> in the order $Li > Na > K > Rb > Cs$; the liquid limits of these clays decrease in the same order. On the other hand, with non-expanding-lattice clays of the kaolinite type, the liquid limit will often <u>increase</u> with the above order of exchangeable ions.

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a. Proc. Paper 2000, April, 1959, by I. Th. Rosenquist.

1. Associate Prof. of Chem. Eng., Associate Director, Soil Stabilization Lab., Massachusetts Inst. of Technology, Cambridge, Mass.

The reduced interlayer swelling with increasing atomic number of the alkali ions can, as the author points out, be explained in terms of increasing polarizability of the cation. It is, perhaps, somewhat more informative to say that swelling increases with increasing hydratability of the cation. In the dry state, the individual lattice-layers of the swelling clays are often assumed to be bonded to one another by both secondary valence-force interactions between adjacent surfaces, and "bridging" via exchangeable cations. In contact with water, the degree to which the layers will separate will depend primarily upon the affinity of the cations for water relative to their affinity for the layer-surface. Thus if the hydration-tendency of the ion is high, water molecules will "cluster" around the ion in large numbers and reduce its intersheet-bridging ability, with consequent high water sorption and swelling. It seems more than coincidental that the hydration energies of the alkali ions increase in exactly the same order as the degree of swelling of the corresponding montmorillinoids.

Inasmuch as the liquid limit is a rough indication of the quantity of mobile or "free" water necessary to provide a lubricating film between adjacent particles in a soil mass, it seems only reasonable that the greater the amount of water taken up into the interlayer spaces of an expanding-lattice clay particle the greater will be the total quantity of water required to provide an arbitrary degree of lubrication between particles; hence the increase in liquid limit with increasing lattice-expansion is to be expected.

With non-swelling clays, the reverse-order of variation of the liquid limit with exchangeable cations seems at first glance to be in disagreement with the above hypothesis. This disagreement is more apparent than real, however. In nearly saturated, nonswelling clay masses, only a small fraction of the total water present appears to be entrapped or immobilized by sorption on the solid; under these circumstances, the existence of a yield value in such a mass must be attributed primarily to interparticle attractions, rather than to a deficiency of lubricant. If this postulate is permitted, then those (high-hydration-energy) exchangeable cations which promote large interlayer swelling in montmorillinoids would, by the same token, also be expected to reduce particle attractions in non-swelling clays, thus favoring low yield-values or low liquid limits.

2. Effect of Electrolytes on the Properties of Clay Suspensions

The author calls attention to the fact that the consistency of clay pastes is markedly altered by the addition of certain electrolytes (e.g. sodium carbonate, silicate, phosphate, etc.), which he attributes to changes in "electro-chemical saturation". He also points out that the leaching of salts from clay masses is usually accompanied by an increase in the electrokinetic potential of the particles. It appears to this writer that the term "electro-chemical saturation" requires clarification, and that the causes of changes in clay consistency and surface-potential with variations in solution composition must be accounted for, since these factors bear very heavily upon the ultimate deductions regarding clay cohesion and structure.

At the outset, it seems cogent to note that the characteristic (electronegative) potential of virtually all clay minerals in aqueous media probably arises from three major sources: (1) isomorphous substitution within the crystal lattice, (2) ionization of weakly acidic (e.g., hydrous silica and

alumina) hydroxyls at the particle surface, and (3) adsorption of anions from the ambient solution. In the three-layer silicates of the montmorillinoid or illite type, the first type of site would appear to predominate. Since such sites would be expected to be strongly acidic (i.e., highly ionized), the exchange capacity of these minerals should be relatively independent of pH; the acid-base titration curves for bentonite seem to confirm this. In the two-layer silicates such as kaolinite, however, both the first and second types of site are to be found, the latter in particularly high-surface concentration; consequently, the cation exchange capacity of kaolinite is to be expected to increase greatly with increasing pH—as indeed it does. Both types of mineral appear to be capable of adsorbing certain anions; this will be dealt with in more detail below. The conclusion to be drawn from these observations is that, with the montmorillinoids, the surface-charge density, and hence the true surface potential is relatively uninfluenced by the ionic environment, while for kaolinite and its two-layer congeners, the surface potential will increase markedly with pH in alkaline environments. Of course, the effective (electrokinetic) potential in the neighborhood of a clay-particle surface is always considerably less than the surface potential because of the presence of counterions in the diffuse ionic atmosphere surrounding the particle; as the classical Gouy-Chapman theory demonstrates, the electrokinetic potential is extremely dependent upon the counterion valency and the ionic strength of the ambient solution. It therefore follows that any variation in electrokinetic potential (and attendant particle-interaction) resulting from a change in solution —environment must be analyzed in terms of alterations in (a) surface-charge-density, (b) type of counterion, and (c) ion-concentration in solution. To ignore any one of these may lead to a distorted interpretation of the facts. Thus, the deflocculation of a clay slurry by the addition of soda ash cannot arbitrarily be attributed to the adsorption of carbonate ion by the clay; this electrolyte will also increase the pH of the slurry, and may therefore increase the surface charge-density, and may also remove polyvalent metal ions (e.g., Ca, Mg) from the double-layer and precipitate them as the carbonates.

Perhaps the greatest insight into the electrokinetic behavior of the clay minerals has been provided by the monumental work of Schofield and Samson,[8] supported by that of Thiessen,[10] van Olphen,[11] and more recently, by Street and his coworkers,[9] demonstrating that despite the dominating negative potential of montmorillinoids and kaolinite, there exist cationic sites on the particles under normal environmental conditions. These sites, located on the flake-edges and attributed to exposed aluminum ions, appear to be responsible for "card-house" (edge-to-face) flocculation of clays, and appear to be the cause of anion-adsorption by these minerals. In the face of these facts, we are compelled to consider a fourth variable in explaining the influence of electrolytes on clay behavior—namely, the interaction of these cationic sites with ions present in the surrounding fluid. For example, in a recent study of the interaction of kaolinite with polyphosphates, this writer[7] has presented evidence to show that the potent dispersive action of these compounds is primarily due to their adsorption on the clay-flake edges via coordination with aluminum. Indeed, it appears likely that most of the very active clay-deflocculants function in this fashion.

In view of the foregoing, it follows that such important engineering properties of clays as paste-consistency, shear strength, and Atterberg limits will depend on the pore fluid composition because of its influence upon the following surface characteristics of the mineral particles:

a.) The surface charge-density; this is dependent primarily on hydrogen-ion concentration (pH).
b.) The thickness of the diffuse double layer; this decreases with increasing electrolyte concentration and increasing cation valence.
c.) The presence or absence of cationic edge-sites on the clay particles; formation of such sites is favored by reduced pH and low salt-concentration in solution, and discouraged by elevated pH, high salt concentration, and/or presence of adsorbable anions.

3. Cementation Processes in Consolidated Clays

Dr. Rosenquist expresses the belief that interparticle cementation via hydrous oxides is unimportant in geologically young sediments. Some recent data which have come to this writer's attention seem to suggest that cementation may be able to develop in clays over rather short time-intervals: Martin[3] has noted that lithium kaolinite converts to aluminum kaolinite on ageing for a period of 100 days in warm (70° C) water. These observations indicate that the hydrous oxides comprising the clay minerals themselves are quite mobile, and that metallic ions in the pore fluid are able rapidly to deposit as hydrous oxides on the clay surfaces. Granting that cementation involves the transport and redeposition of such oxides at particle contact points, it may well be that the process can occur quite rapidly under suitable (and not particularly extreme) conditions.

4. Effect of Cations on the Sediment Volume of a Clay

Dr. Rosenquist cites measurements of the sediment volume of clay in solutions of Li, Na, K and Cs chlorides, and of aniline and cyclohexylamine hydrochlorides, as evidence of bond-strengths between particles. In this writer's opinion, such measurements reveal very little about bond-strengths per se; only those bonds which are weak enough to fail under the small gravitational forces acting on the sediment will succumb upon sedimentation, and bonds of this magnitude are probably of minor importance in determining cohesion. A more convincing measurement revealing interparticle bond strength would perhaps be the slope of the pressure-vs-consolidation curve for these sediments.

On the other hand, the sedimentation volumes tell us a great deal about the geometric arrangement of particles within the flocs which comprise these sediments. As a matter of fact, Dr. Rosenquist's observations on a naturally occurring clay are in remarkable accord with the theory and data of Schofield and Samson (loc. cit.) as determined with pure kaolinite. In 0.5 normal solutions of the alkali halides, they show that cationic edge-charges are neutralized by chloride ions, and that the (negative) zeta potential of the placelet faces is reduces by the high salt concentration; as a consequence, edge-to-face association of particles is prevented, and the particles will tend to remain separate, or cluster into dense "card-pack" agglomerates, depending on the degree of zeta-potential suppression. Since the higher-atomic-number alkali ions have (because of reduced hydratability or increased polarizability) a greater ability to reduce the zeta potential, the degree of flocculation increases in that order. Since a suspension of independent platelets can be expected to occupy a smaller volume on sedimentation than one of "card-pack"

agglomerates, the observed variations of sediment volume with atomic number of the alkali ions are what might be expected. Indeed, one would expect that the minimum quantity of solution required to render a particular clay fluid (and thus, presumably, the liquid limit) would increase directly with the sediment volume; hence, the afore-mentioned observations suggest that the liquid limits increase in the order Li < Na < K < Cs, as anticipated in (1) above. It should be added, however, that the particle arrangements in these sediments is likely to be strongly influenced not only by the nature of the exchangeable cation, but also by the total salt concentration. For example, extensive card-pack agglomeration of a lithium clay can probably occur in very concentrated lithium chloride solutions, while in very dilute potassium chloride solutions, a potassium clay may be rather well-dispersed. Under these circumstances, the sediment volume of the K-clay can actually be less than that of a Li-clay. The importance of the concentration of salt in the pore-fluid of a clay in determining particle packing-geometry hence cannot be overemphasized.

The behavior of the amine hydrochlorides is quite different from that of the alkali halides, and for good reason. What has been overlooked by the author is that both aniline and cyclohexylamine hydrochloride are salts of weak bases, which hydrolyze extensively in aqueous solution, literating free hydrochloric acid. Since aniline is a far weaker base ($K = 4.6 \times 10^{-10}$), than cyclohexylamine ($K = 4 \times 10^{-4}$), the pH of an aniline hydrochloride solution will be lower than that of cyclohexylamine hydrochloride. Recalling that the number of cationic edge-sites on a clay has been shown to increase with decreasing pH, one would predict extensive edge-to-face flocculation in these amine salt solutions, and more with the aniline salt. Since edge-to-face flocculation yields very low-density, "card-house" flocs, the high sedimentation volumes observed are only normal. This, of course, is not to be construed to indicate that the amine ions play no part in the process; to the contrary, they adsorb far more extensively on the platelet faces than do the alkali metal ions, and reduce the zeta potential more markedly.* This will permit face-to-face flocculation to occur if two particles are brought into sufficiently close proximity, but it can easily be shown that the most probable initial contact between two plates in free motion will be edge-to-face. Hence, formation of card-house flocs will be most likely in this situation.

The ultimate conclusions of both the author and this writer are, nonetheless, the same: any environmental change which either reduces interparticle repulsions or increases interparticle attractions will increase the adhesion and shear strength of the clay, and vice-versa.

5. The State of Adsorbed Water on Clays

The fact that the heat of adsorption of water by clays is of a magnitude roughly equal to the heat of sublimation of ice, is of itself no indication that the organization of water molecules in proximity to a clay surface is comparable to that in ice, nor that such water is "solid". Even proof that the density of adsorbed water on clays is lower than that of liquid water at the corresponding temperature is not unequivocal evidence that the adsorbed water is

*The effect arises not so much from the "polarizability" of the ring-constituents, but rather from the fact that the rings possess little dipole-moment, and are far less attracted to water molecules than water molecules are to one another.

"ice-like"; indeed, Low and his coworkers[1,2] have demonstrated that adsorbed water on clays supercools much more extensively than liquid water, suggesting that adsorbed water is definitely not oriented in an ice-like configuration. There can be little doubt, however, as Dr. Rosenquist states, that the physical state of adsorbed water is different from that of "normal" water; this writer would like to add the postulate that such adsorbed water is not only "different", but anisotropic—that is, the adsorbed layer possesses physical properties in a direction normal to the surface different from those measured in the plane of the surface. Thus, it would seem reasonable that the resistance offered by a water molecule being slid along the surface may be considerably less than that offered by a water molecule being elevated from or lowered toward the surface. It therefore appears to this writer dangerous to attempt to draw conclusions regarding the "viscosity" of adsorbed water from thermodyamic data such as heats of adsorption, or partial modal volumes in the adsorbed state. It might be added here that a great deal of data have been obtained on gaseous diffusion through microporous solids which indicate that much of the gas flow occurs by lateral transport of molecules in the adsorbed layers; this presumably means that the adsorbed gas molecules are far from "immobile" in the two-dimensional sense.

Dr. Rosenquist's observations regarding the rate of equilibration between a clay and heavy water (D_2O) are in virtual accord with, and almost identical in nature to, those of McAuliffe et al.,[4] published in 1947. However, these measurements indicate only the net rate of exchange of protons (or deuterons) between the phases, and throw little if any light on water-mobility per se. It will be recalled that the self-diffusion coefficient for hydrogen ions in water is far higher than that of any other ion, from which it has been deduced that protons move through water by a chain-transfer process much the way electricity is conducted through metals. Since there need be no net transport of oxygen atoms for proton- or deuteron transport to occur in water, it seems doubtful to this writer that deuteron-transport measurements can throw much light on the mobility or viscosity of adsorbed water.

Another complication which Dr. Rosenquist does not call attention to in his discussion of his deuteron-diffusion measurements in clays is the problem of estimating the effective path-length for diffusion through a porous solid. It has been well-demonstrated by Wyllie and Spangler[12] and others that the "tortuosity" of the flow (or diffusion) path in a porous solid is appreciable, and can become exceedingly large in oriented masses of anisometric (e.g., platy) particles. This tortuosity factor will create the impression that the diffusivity of a substance through such a solid is far lower than its true diffusivity in the flow channels. Thus, Dr. Rosenquist reports diffusivities of the order of 0.15 cm^2/day in clays, versus 3.0 cm^2/day in liquid water. This corresponds to a tortuosity factor of about 20, which seems not at all unreasonable for a consolidated clay mass. Hence, even if deuteron-transport measurements did reflect water-mobility or viscosity, measurements made in clays could not be interpreted in terms of changes in water mobility unless the tortuosity factor were precisely known. Unfortunately, there are no suitable techniques for determining this factor with accuracy, except in geometrically very simple systems.

It appears appropriate at this point to refer to the work of Michaels and Lin,[5] which Dr. Rosenquist cites, but unfortunately attributes to these authors conclusions which they did not draw. They found that the permeability

of dry kaolinite to nitrogen gas was only slightly greater than that of the clay to liquid water, provided the clay mass was consolidated initially in water. From this, they deduced that there was very little abnormally viscous or "immobile" water in wet clay, and that if an "immobile" film existed, it was probably no more than 20 molecules thick. In a subsequent publication,(6) it was shown that a good part of the difference between water-and-nitrogen-permeabilities in kaolinite was due to counter-electrosmotic effects, thus rendering even less likely the existence of an immobile water film of significant thickness.

We are, therefore, compelled to conclude that evidence supporting the presence of viscous or solid water on the external surfaces of clay minerals (insofar as its ability to withstand shear stress without movement is concerned) is still inconclusive; and hence, we feel that a satisfactory explanation of cohesion in clays must avoid reliance upon the concept of viscous or solid adsorbed water as a major cohesion-contributing factor.

By the same token, it appears difficult to this writer to find reason for attributing cohesion to the existence of an interfacial tension between adsorbed water and "free" water in the voids. How can such an interface exist? Between any two homogeneous phases in equilibrium, there can be but one interface, and but one interfacial tension, no matter how thick the interfacial zone may be. As Gibbs has pointed out, one may locate the "surface tension" between two phases any place he chooses, but this does not alter the fact that the tension exists only because the two phases are in contact.

Certainly, there is free surface energy at the clay-water boundary, and for this reason there is a driving force for cohesion between particles in order to reduce this free energy; but it does not seem particularly illuminating to assign this driving-force to a rather nebulous region within the boundary layer where no clear-cut discontinuities have been detected.

6. The Role of Water in Clay Cohesion

The proposition that water functions as a "molecular glue" between clay particles and is directly responsible for cohesion, seems to this writer indefensible, not only on physico-chemical grounds, but also in the light of the facts. The most important of the latter are:

a.) With cohesive soils and clays, provided precautions are taken to prevent cracking on shrinkage, there is no evidence (to this writer's knowledge) to show that the cohesive strength vanishes or drops to low values when the last traces of adsorbed water are removed.

b.) Dehydration of a water-containing cohesive clay by solvent-replacement yields a dry solid with a cohesive strength roughly equal to that of the initial wet mass.

c.) The dry-strength (and density) of a deflocculated clay is much higher than that of a flocculated clay, despite the fact that, in the water-wet state, the former is much weaker than the latter.

The physico-chemical counter-arguments are perhaps even more convincing: Why do clays (or, for that matter, all polar solids) adsorb water in the first place? The most tenable explanation is that there exist at the solid surfaces unsatisfied molecular force-fields (they may be Van der Waals, dipolar, ionic, or covalent—at this point, their nature is unimportant) into

which the water-molecules are drawn. What effect must the adsorptive process have upon these residual forces? It must reduce them; indeed the universal reduction in sorption-energy with extent of sorption is proof of this. If the surface forces are reduced by adsorption, how, then, must the forces of cohesion or adhesion between neighboring solid surfaces be altered? Certainly, they too must be reduced.

It is often argued that, since water is dipolar, and since the surfaces of clays and related minerals are ionic checkerboards, water molecules can orient outward from the surfaces and form chain-like links between particles, thereby binding them together. Granted that this is a reasonable postulate—does this mean that water is primarily responsible for binding between particles? Hardly—this would be equivalent to arguing that two bar-magnets which are separated, but surrounded by iron filings, are held together by the filings, when it is obvious that the binding force resides solely in the magnets themselves.

In the presence of a swarm of dipoles, however, it can be argued that attraction between two dipolar surfaces separated by a relatively large distance will be greater than that between the two surfaces in the absence of the dipoles. (It is this effect which is responsible for the high dielectric constant of substances such as water.) On the other hand, at short distances between surfaces, the force of attraction will be reduced. One therefore might expect that the presence of water around clay particles could allow the particles to exhibit weak attractions for one another at rather large interparticle distances, (say, 10-50 Angstroms) although the water would reduce strong attractions at short distances (less than 10A°). The net consequence of the presence of water should, nevertheless, be a general reduction in particle-adhesion.

On a basis of the foregoing, then, one may surmise that interparticle-adhesion in clays occurs in spite of, rather than because of, the presence of water. We must therefore look to some other function of water to explain why it is that a compacted dry, powdered clay exhibits little cohesion or plasticity, whereas wet clay does so.

The answer to the problem may lie in the long-range coulombic or electrostatic forces which develop at mineral surfaces in high-dielectric-constant media such as water, the dependence of these forces upon the composition of the water phase, and the way in which these forces guide or direct the geometric arrangement of primary particles. As Dr. Rosenquist so clearly points out, the most probable first contact between plate-shaped particles in free motion is between a corner or edge, and a face. If adhesion can occur at these points of contact, then the particles will be immobilized in this configuration; this does not necessarily mean that this configuration is the most stable one. For example, in a dry clay, it is probable that all surfaces of the primary grains are capable of adhering to one another. If one therefore takes a cohesive dry clay mass in a card-house structure (prepared, for example, by organic-solvent extraction and drying of an undisturbed wet clay) and crushes it, collapse of the card-house is to be expected, with the formation of small, randomly arranged packets of sheets in predominantly face-to-face arrangement. These packets (containing many thousands of individual particles), being held together by surface forces acting over a relatively large contact-area, function as rigid solid entities; efforts to force these packets together into a coherent mass are ineffective, not because the packets cannot stick when brought into contact, but rather because the available area of

packet-contact per unit volume of clay is vanishingly small compared with that existing in the undisturbed card-house state.

"Card-House" - Undisturbed Dry Clay (Cohesive)

"Packets" - Disturbed Dry Clay (Non-Cohesive).

If we assume that, in a card house structure, each platelet must contact at least two others to yield a coherent structure, then there is, on the average, one contact point per primary particle. In a collapsed, packet structure, we can similarly assume that there must be one contact point per packet for coherence; if there are a thousand particles per packet, then clearly there will be one thousandth as many contact points per unit volume (at the same density of compaction) for the packet-structure as for the card-house structure. Thus the absence of cohesion in compacted, dry clays is hardly surprising.

In the presence of water under normal conditions, it has been shown that coulombic or double-layer forces tend to cause repulsion between platelet faces, and to cause attraction between edges and faces. Furthermore, these forces are active over large distances on a molecular scale—hundreds or thousands of Angstrom units. If primary clay particles are dispersed in water, these forces will direct the particles into a card-house structure, and furthermore this structure will be stable as long as water is present. If we attempt to disrupt this structure by mechanical means, we will meet resistance to such disruption, since the edge-face attraction must be overcome to alter the packing-geometry. If the forces we apply are great enough to overcome these attractions, we will deform the system, and the particles will move with respect to one another, but on removal of the stress, the particles will occupy new positions of stable attraction. As a consequence, the mass will be both cohesive and plastic. The loss of plasticity of a clay on desiccation without alteration of particle packing geometry or structure (e.g., by solvent-extraction) can be explained simply on the basis that, once the interparticle adhesive bonds between adjacent particles are broken by application of sufficient external shear force, particles in the shear plane, once liberated from their opposite neighbors, collapse upon one another into packets, thereby greatly reducing the available bonding area at the shear plane.

In essence, the foregoing explanation of clay cohesion and plasticity is based on only two assumptions: (1) that spontaneous adhesion between particles will occur if they are brought into sufficiently close proximity, and (2) that the geometric arrangement of the particles, which determines the frequency of interparticle contacts, is the primary strength-determining variable. Such an explanation avoids the necessity of making assumptions as to the nature of adsorbed water, or to the role of electrolytes and exchangeable ions in influencing interparticle adhesion. As pointed out earlier, it is

relatively easy to show how the aqueous phase composition and the type of exchangeable ion influences the packing-geometry of clay particles; specification of the geometry is itself sufficient to determine (qualitatively, at least) many macroscopic properties of a clay mass.

To ascribe cohesion and plasticity of wet clay masses exclusively to particle adhesion and geometry would obviously be an over-simplification of the phenomenon. There are clearly other "binding forces" which come into play in many, if not all clay systems. As Dr. Rosenquist notes, many saturated soils (not only sands and silts, but also many clays) which exhibit significant shear strength in air, will rapidly disintegrate on immersion in water. Cohesion of this sort certainly cannot be ascribed to interparticle adhesion, since it is inconceivable that such bonds should be broken by mere immersion in water of a mass already completely saturated. "Cohesion" of this sort is undoubtedly attributable to capillary pressures developed at water-air menisci present on the surface of the sample. If a mass of particles which exert absolutely no attraction at all for one another are compacted in the presence of water, they will tend to assume a packing configuration corresponding to minimum porosity. If this saturated mass is subjected to distortion in air, an increase in void-volume will result, thus drawing water from the surface to the interior of the mass. This in turn will cause formation of curved air-water menisci at the sample boundaries, and development of hydrostatic tension in the water phase. This tension manifests itself as a resistance to distortion, or cohesion, and persists under load whether the mass is permanently deformed or not; consequently, the wet solid appears to be both "cohesive" and "plastic". Since these properties result, not from the solid phase, but from the pore fluid exclusively, it may be preferable to refer to them as "hydrostatic cohesion" and "hydrostatic plasticity" respectively. Since the magnitude of the pore-water tension which can be developed by distortion of a basically non-cohesive sediment is inversely proportional to the mean pore-diameter, it follows that "hydrostatic cohesion" increases rapidly with decreasing particle size; in non-swelling clays such as kaolinite, where pore diameters of 2000 A° are not uncommon, the "hydrostatic cohesive strength" may be in the neighborhood of 100 psi, which can represent a large contribution to the observed compressive strength.

In partly-saturated clay or soil masses, the contribution of capillary tension to apparent cohesion can become very great indeed, simply because of the fact that as saturation is reduced, the water-air menisci retreat into progressively smaller pores where the water-tension progressively increases. Increase in apparent cohesion will continue with reduction in saturation until finally there is insufficient "free" water present to form annular lenses at the particle contact points, whereupon the mass should become non-"cohesive", and disintegrate. This phenomenon is, of course, observed with relatively coarse granular solids but not with even deflocculated clays; one tenable reason for the fact that clays do not disintegrate on drying is simply that interparticle adhesion resulting from removal of adsorbed water (producing <u>true</u> cohesion) begins to develop during drying long before all the menisci have disappeared. The observation that certain cohesive (non-swelling) clays expand in volume as their water content is reduced below a quite small value (e.g., 5%) suggests that capillary tensile forces may be important contributors to strength even in rather dry clay masses. It must therefore be clearly recognized that a general theory of cohesion and plasticity in soils must consider both (1) particle-interactions, and their dependence upon environment

and particle geometry, and (2) pore fluid-gas boundary characteristics, and their dependence upon particle size, geometry, and degree of void-saturation. Application of these criteria to soils is helpful in explaining not only cohesion and plasticity in clays, and other colloidal solids, but also the absence of these properties in other consolidated granular solids.

In summary, it can be stated that we are in complete agreement with Dr. Rosenquist on the following points:

1.) Van der Waal's attractions between clay particles are of a magnitude more than adequate to account for cohesion in clays.
2.) The degree of, or propensity toward interparticle adhesion in clays is markedly dependent upon the composition of the pore fluid, and the nature of the exchangeable cations.
3.) Water in the immediate vicinity of a clay particle surface possesses physical properties different from those of bulk liquid water.
4.) The strength of a consolidated clay is controlled to a great extent by the volume-density of interparticle contacts.
5.) The common arrangement of particles in a cohesive clay mass is a "card-house" structure.

Major points of interpretative disagreement are as follows:

1.) Adsorbed water is not, we believe, responsible for inter-particle adhesion, but rather discourages it from taking place.
2.) There is not, in our opinion, much convincing evidence to show that adsorbed water on clays is either solid or highly viscous.
3.) The exchangeable ions and the electrolytes in the pore fluid do not, we believe, play much of a direct role in promoting interparticle adhesion. Their function, we feel, is to alter the long-range coulombic forces around the clay particles, which, in turn, direct the arrangement of particles within the consolidated clay mass.

Finally, we feel that capillary tension in the pore fluid is an important contributing factor to soil cohesion, not only in coarse, granular soils, but also in heavy clays. Due consideration of this factor must be taken in order to explain adequately the cohesion-plasticity phenomenon.

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CONTENTS

DISCUSSION

Page

Mississippi Valley Geology—Its Engineering Significance, by C. R. Kolb and W. G. Shockley. (Proc. Paper 1289, July, 1957. Prior discussion: 1319. Discussion closed.)
by C. R. Kolb and W. G. Shockley (closure) 105

Model Study of a Dynamically Laterally Loaded Pile, by Roy D. Gaul. (Proc. Paper 1535, February, 1958. Prior discussion: 1828. Discussion closed.)
by Roy D. Gaul (closure) 107

Cement and Clay Grouting of Foundations: The Use of Clay in Pressure Grouting, by Glebe A. Kravetz. (Proc. Paper 1546, February, 1958. Prior discussion: 1828. Discussion closed.)
by Glebe A. Kravetz (closure) 109

Cement and Clay Grouting of Foundations: The Use of Admixtures in Cement Grouts, by Alexander Klein and Milos Polivka. (Proc. Paper 1547, February, 1958. Prior discussion: 1828. Discussion closed.)
by Alexander Klein and Milos Polivka (closure) 115

Cement and Clay Grouting of Foundations: Suggested Specifications for Pressure Grouting, by Judson P. Elston. (Proc. Paper 1548, February, 1958. Prior discussion: 1828. Discussion closed.)
by Judson P. Elston (closure) 117

Cement and Clay Grouting of Foundations: Experience of TVA with Clay Cement and Related Grouts, by George K. Leonard and Leland F. Grant. (Proc. Paper 1552, February, 1958. Prior discussion: 1828. Discussion closed.)
by George K. Leonard and Leland F. Grant (closure) 121

(Over)

Note: Paper 2011 is part of the copyrighted Journal of the Soil Mechanics and Foundations Division, Proceedings of the American Society of Civil Engineers, Vol. 85, SM 2, April, 1959.

Page

Increased Resistance to Deformation of Clay Caused by Repeated Loading, by H. B. Seed, R. L. McNeill and J. DeGuenin. (Proc. Paper 1645, May, 1958. Prior discussion: 1881. Discussion closed.)
by H. B. Seed, R. L. McNeill and J. DeGuenin (closure) 125

Geological Factors in Tunnel Construction: Tools and Techniques, by A. B. Cleeves. (Proc. Paper 1648, May, 1958. Prior discussion: 1881. Discussion closed.)
by A. B. Cleaves (closure) 127

Geological Factors in Tunnel Construction: Geology and Tunnel Design, by A. B. Reeves. (Proc. Paper 1649, May, 1958. Prior discussion: 1881. Discussion closed.)
by A. B. Reeves (closure) 129

Design and Performance of Vermilion Dam, California, by K. Terzaghi and T. M. Leps. (Proc. Paper 1728, August, 1958. Prior discussion: 1951. Discussion closed.)
by R. W. Spencer 131
by P. J. West. 132
by J. H. Birman. 136
by B. R. Laverty 137

Procedure for Rapid Consolidation Test, by Hsuan-Loh Su. (Proc. Paper 1729, August, 1958. Prior discussion: 1959. Discussion closed.)
by Carl W. Fenske 145

Computation of the Stability of Slopes, by Otto H. Meyer. (Proc. Paper 1824, October, 1958. Prior discussion: none. Discussion closed.)
by G. S. Sarkaria and N. L. Worth. 147

Glossary of Terms and Definitions in Soils Mechanics by the Committee on Glossary of Terms and Definitions in Soil Mechanics. (Proc. Paper 1826, October, 1958. Prior discussion: 1951. Discussion closed.)
by John A. Focht, Jr. 151
by Bernt Jakobson 153
by R. G. Ahlvin 159
by Paul A. Parisi. 159

MISSISSIPPI VALLEY GEOLOGY—ITS ENGINEERING SIGNIFICANCE[a]

Closure by C. R. Kolb and W. G. Shockley

C. R. KOLB[1] and W. G. SHOCKLEY,[2] M. ASCE.—Mr. Li's kind remarks are very much appreciated and his account of geologic conditions in the Atchafalaya Basin is a valuable supplement to our discussion of Mississippi Valley geology and its significance to engineers. The senior author was fortunate to have played a considerable part in the geological investigation of the Atchafalaya Basin and the problem of Mississippi River diversion (items 11 and 12 in Mr. Li's list of references). Hence he feels qualified to fully indorse Mr. Li's summary of geologic events which shaped the basin and which affect the diversion process.

However, one point should be discussed further. Mr. Li states that "Even after completion of [the necessary engineering works to control diversion,] there is no eternal guarantee that a major breach would not happen, should untenable conditions due to climatic abnormalities occur, for the control works are located in an alluvial plain." While no "eternal guarantee" is possible there is good evidence to indicate that past diversions of the Mississippi River have occurred only when certain specific and unusual conditions have been met. Early stages of past diversions appear to have been gradual and unspectacular, potential diversions often sealing themselves. It is only when the diversion channel is taking low-water flow that it is capable of continued enlargement, and only when it has developed to the point where it is taking some 40 per cent of the flow of the master stream that the process accelerates. Available evidence indicates that this critical stage (40 per cent diversion) takes as much as half a century. The point is that sudden changes in course of the Mississippi are an extremely remote possibility. Normal diversionary development is so slow that engineering works can be constructed in plenty of time to prevent total diversion. The size and cost of such structures, moreover, can be significantly reduced when the geologic criteria that indicate and control diversion are understood and recognized at an early stage.

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a. Proc. Paper 1289, July, 1957, by C. R. Kolb and W. G. Shockley.

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MODEL STUDY OF A DYNAMICALLY LATERALLY LOADED PILE[a]

Closure by Roy D. Gaul

ROY D. GAUL,[1] J. M. ASCE.—It is indeed gratifying to receive such detailed discussion from men as well versed in the subject of lateral loading on piling as Messrs. Reese, Matlock and Focht. Their criticism adds considerably to the value of this experimental study.

The opinion by Reese and Matlock "that continued cyclic loading, say at the 91 pound load, would have caused a large increase in the magnitude of the bending moments" deserves some attention. Continuous cyclic loading was applied for a period of several minutes before each data "run" and the cycling was stopped only long enough to increase the stroke adjustment for each loading during a test series. At no time were fluctuations or gradual increases in strain gage outputs noted. This should be expected (discounting the possibility of resonance) providing that at no level along the pile inelastic soil behaviour occurs. That such was the case up to loads of about 50 pounds is discussed extensively by Mr. Focht and illustrated in his Fig. 1. Consequently, it is legitimate to regard the parameter "k" as a function of depth only for loads of 50 pounds or less.

A prototype load of 34400 pounds is equivalent to the 91 pound static load on the model and a stress of about 9200 psi would be produced in the 36 inch steel pile. Therefore, concern about the soil-pile characteristics in the higher range of loads is certainly merited but the model testing program was designed for study of elastic behavior only. The writer concurs that original conclusions based on comparison of these dynamic and static test results should specify elastic soil conditions as a limiting factor.

Pile deflections at the mudline were approximately 0.25 inches when separation began to occur during both static and dynamic loading. There was some difficulty in locating a "zero" reference point for static loads but it is estimated that horizontal deflection at the mudline produced by the 91 pound load was between 0.4 and 0.5 inches.

Apparently greater emphasis in the original paper should have been placed on the restrictions inherent in the model study. Its primary purpose was to investigate dynamic stress amplification and phase relationships of the oscillatory load to pile vibration within elastic ranges of soil stress. Failure to specifically limit the conclusions originally stated have elicited justified criticism, especially as regards application of the results to field conditions. Not until a wide range of model and/or prototype testing programs have been completed can general relationships be confidently formulated.

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a. Proc. Paper 1535, February, 1958, by Roy D. Gaul.

1. Oceanographic Engr., Marine Advisers, La Jolla, Calif.

CEMENT AND CLAY GROUTING OF FOUNDATIONS: THE USE OF CLAY IN PRESSURE GROUTING[a]

Closure by Glebe A. Kravetz

GLEBE A. KRAVETZ,[1] A. M. ASCE.—The author appreciates the discussion by Mr. B. E. Clark and welcomes this opportunity to comment on a few points of interest raised by the discusser.

1. Permanency of Grouts

A grout has to be permanent, only as long as it is needed. For instance, in a grout curtain under a dam the useful life of a grout should be counted in decades, while in a grout curtain under an upstream cofferdam its useful life can be counted in years, or, if the grout is used to stop quicksand conditions during the driving of a tunnel, its useful life need be only a matter of weeks. Also, a grout with a shorter life than the structure can be preferred to a more permanent and more expensive grout if maintenance or corrective grouting is found to be economically feasible. Furthermore, the useful life of a grout may vary with the conditions under which it is used. For instance, the use of a water soluble chemical grout can be objectional if ground circulation is relatively large, but will be quite acceptable if the circulation is insignificant.

Clay grouts, although they never reach the consistency of a stiff clay, have nevertheless adequate rigidity to withstand an appreciable amount of water circulation without any significant washing or leaching. Besides, the piping conditions which exist within a grouted foundation are radically different and much less severe than the conditions existing at the bottom of a single wall cofferdam, or at the downstream toe of an earthdam where the displaced material moves into open space.

Also, the most critical conditions within the foundation are normally created at the time of the grouting, and it is difficult to see how, afterwards, the clay particles in a grout curtain could readily migrate. Such a condition could exist only if the clay grout was improperly selected, low pressures were applied during the pumping or, in a grouting program including several types of grouts, the proper sequence of grouting had not been followed. Clay grouts, it should be added, very rarely have been used alone. Usually the foundation conditions are such that clay grout is used together with other grouts. A classic example by now of such a grouting job was the grouting, in 1943, of the Digue du Lac Noir[2] where four different types of grout were used: clay-sand-

a. Proc. Paper 1546, February, 1958, by Glebe A. Kravetz.

1. Soil and Foundation Engineer, Tippetts-Abbett-McCarthy-Stratton, Engineers, New York, New York.

2. Ischy, E. Lutte contre les erosions souterraines du Lac Noir. Proc. 2nd Int. Conf. Soil Mechanics and Foundation Eng. Ix.c.

cement, cement, clay and sodium silicate. In this grouting program, like in any properly designed and executed program, the water circulation has been reduced to the extent that the clay grout cannot be expected to be washed away. However, should this happen, the grout curtain is so designed that the clay will migrate towards the central core of the curtain which consists of a more permanent grout. At this core the clay will stop and gradually accumulate.

It is the author's belief that too often the danger of washing or piping of a clay grout is overstressed and that clay grout curtains are sometimes overconservatively designed, either by using an unnecessary amount of more permanent material such as chemical or cement, or by making the curtain excessively wide.

2. Strength of Grouts

High strength grouts are generally not needed. It is only in a few special cases such as the grouting of old masonry structures and tunnel linings or the making of concrete by the Colcrete, High-Turbulance or Prepakt methods that high-strength cement grouts are required. In rock foundation grouting, ordinary neat cement, or clay-cement grouts have adequate strength, even for instance, to grout for consolidation grouting jointed rock abutments for arch dams. The purpose of consolidation grouting is not to reinforce the rock by injecting some high strength material, but rather to make the whole rock mass monolithic and thus prevent some local displacement. This is why, in fact, the adherence of a grout to the rock is very often more important than its strength. In soil foundation grouting, the grouts used usually have very little or no strength at all, per se. If consolidation is sought, it is the resultant strength of the grouted foundation material which is important: for instance, both a sodium silicate gel and a loose sand have, separately, very little strength, while the same sand after it is grouted will have the strength of a soft sandstone. If only impermeabilization is sought, the grout strength is unimportant. For example, a sand and gravel foundation has usually more than adequate bearing capacity and does not need any additional strength.

Clay grouts need only enough rigidity (or strength) to resist washing. Clay-chemical grouts have a strength comparable or higher than the chemical grout itself. Depending on its clay-cement ratio, the strength of a clay-cement grout can be made to vary greatly. For a comprehensive study of the strength of clay-cement grouts the reader is referred to References #1 and #9 of the paper.

3. Adherence of Grouts

The fact that in rock foundation grouting adherence of the grout to the rock may be more important than its strength has already been mentioned above. In his paper, the author refers first to the adherence of clay-cement grouts to ". . . clay coated walls of cracks and faults in rocks . . ." and a second time to the adherence of clay-sand-cement grouts to rock. That these grouts have a better adherence or compatibility to rock than ordinary cement grouts is, by now, a well-established fact. The reasons offered for this are:

1. Most probably and very simply better adherence is due to the fact that clay containing grouts usually have a V_C/V_S ratio close to 1. In other words, they shrink much less than ordinary neat, cement grouts.
2. The presence of fine clay grains and usually better dispersed cement grains (clay-cement grout requires a more energetic mixing than ordinary cement grout) facilitates the penetration of the grout into the rugosities of the opening in the rock.
3. There is a possible affinity between the clay particles of the grout and the clay coating the walls.

While these reasons may be controversive, the use of bentonite clay as an additive to cement grouts to improve their adherence is becoming more of a general practice. The amount of clay added usually varies from 2% to 14% of the weight of the cement.[3] Good adherence, however, can also be obtained without adding clay to the cement grout, provided that the rock openings are clean or can be thoroughly cleaned by washing and that a non-shrinking cement grout is used. Wetting agents, also, can be added to the wash water to improve its wetting action and the subsequent adherence of the grout.

In soil foundation or embankments, the adherence of grouts is a problem of a different nature, because the subsequent settlements and deformations which usually take place cannot be prevented. Because of these movements, grouts which harden after setting cannot adhere permanently to the soil, while grouts which set in a plastic or semi-plastic state adjust to the gradual deformations. The latter type grout includes among others: asphalt emulsions, sodium silicate and clay grouts.

4. Stability of Grouts

All suspension grouts segregate. For some grouts the rate of settlement of the solid particles can be so fast that only mechanical agitation or turbulent flow of the water will maintain the particles in suspension. Because of the material used (sawdust, rice, chopped straw, sand with or without a supporting material such as cement) these grouts can be defined as coarse grouts. They are used because of their lack of stability to plug voids absorbing unnecessarily large amounts of grout, or in rock core drilling, when large openings through which the drill water is lost are encountered.

In many of the grouts commonly used in pressure grouting, however, the rate of settlement of the particles is comparatively low, so that under average conditions, the grouts can be considered as stable.

The stability of ordinary cement grouts is adequate for rock foundation grouting, but rather low for soil foundation grouting where pressures drop sharply as soon as the grout enters the soil openings. This explains why a sand foundation can be grouted successfully by a clay-cement grout, when it could not be grouted by a cement grout, although both grouts have practically the same groutability ratio (D_{15} soil/D_{85} grout). In both cases the cement penetrates the sand, but because of its low stability the neat cement grout cannot travel far enough to make the grouting practical.

Ordinary sand-cement grouts have a low stability. Those who have experimented with these grouts under field conditions, which sometimes include

3. Galerie d'Ait-Ouarda-Afourer. Barage d'Ait-Ouarda. Report published by Sondages, Injections, Forages, 11 Ave. Colonel Bonnet. Paris XVI. France.

among other things, several hundred feet of cement coated pipes with innumerable elbows, joints, connections, etc., know how easily the pipes can become plugged. Even in the tests mentioned by Mr. B. E. Clark,[4] it is indicated that the sand-cement grout could be circulated (under not so trying conditions) only if careful pumping techniques were followed. The scope of these tests, incidentally, did not cover the study of grout segregation, and the word itself is not even used in the report.

In the paper, the author indicates (page 5) that the stability of a clay suspension can be increased through mechanical, electrochemical and physical action. The stability of the various grouts used in pressure grouting can be improved similarly, either by high speed mixing, peptizing electrolytes, wetting or dispersing agents or colloidal size materials such as clay.

5. Use of Clay Grouts

When selecting the types of grout to be used for a given pressure grouting project, three basic points should be considered:

1. The purpose of the grouting
2. The nature of the foundation material to be grouted
3. Economics

For example, if the purpose of the grouting is consolidation, then depending on the strength, permanency and the other qualities required from the grout, such inert materials as sand and clay can be readily discarded or used only in small proportions. Similarly, if the material to be grouted is a granular moraine deposit requiring impermeabilization, then the use of several grouts such as chemical, clay, clay-cement, clay-sand-cement must be anticipated from the start. If, at last, several grouts such as various asphalt emulsions and chemicals can do the job equally well, then the most economical one must be selected.

A grout, like any other construction material, has its own qualities and limitations and consequently its proper field of use. Thus, careful design, which in pressure grouting calls for the judicious selection and use of a grout, is just as necessary in the planning of a grouting program as in any other engineering work.

Clay is not just a cheap, fine-grained grout material, which is to be used merely because it happens to penetrate openings too small for cement, fill voids too big for chemicals alone, stabilizes suspension grouts or makes cement, chemical or any other grout more economical or more compatible to the foundation material. When the use of clay, either as a clay grout or as an admixture is considered, the adverse effects resulting from its use must be weighed against the resulting advantages as in the case of any other type of grouting material. It can be said, however, that when clay is used as an admixture, the amount by which qualities such as strength, permeability and resistance to leaching are adversely affected can always be kept within limits acceptable for the specific use of the grout and that the increase in grout stability, plasticity and thixotropy are such that the overall quality of the grout is improved.

4. "Tests of Sanded Grouts". Report No. 1. Tech. Memo 6-419. Waterways, Experiment Station, Vicksburg, Mississippi. October 1955.

6. Effect of High Speed Mixing on Cement Particles

High speed mixing affects a cement particle in two ways:

1. Grains of a powdery material, when mixed with water, tend to aggregate and form lumps, thus slowing down the wetting process of each individual grain. Also, gas bubbles remain attached to some of the grains and lumps thus further slowing the wetting process. High speed mixers are usually so designed as to create a shearing or laminating effect. The laminating or shearing effect plus the high speed (1500 to 3000 rpm) and centrifugal effect thoroughly breaks the aggregates or lumps, and separates the gas bubbles from the grains. As a result, each individual grain is rapidly and thoroughly wetted and put into suspension.
2. The crystallization and colloid theories of the cement hydration process are not wholly accepted but the picture of the process itself is fairly well established. During the hydration, needle-like or string-like elements of hydrates form on the superficial layer of each wetted grain of cement. In a high speed mixer, the laminating effect and high speed rotation keep breaking these hydrates away from the grain of cement, thus exposing new areas of the grain to water and consequently bringing the formation of new elements. These hydrate elements are of colloidal size and as the amount of these elements in the mixture increases the cement grout becomes colloidal in character.

Based on the preceding somewhat simplified description of the process which occurs in a mixer, it can be said that:

a) High speed mixing activates the hydration of the cement, or more correctly, the first phase of the hydration of the cement (cement is not fully hydrated even after one year). This activated hydration has, however, no effect or at least no appreciable effect on the setting time of the cement grout.

b) High speed mixing changes the granular composition of the cement. While the last statement may be more a subject of semantics than physics, there is no doubt that the size of a cement grain continuously changes from the moment it is wetted to the very end of the hydration process. At first, its size increases as the hydrates form; then as hydrates are being broken away from the grain and new hydrates develop, the size of the grain of cement starts to decrease. Also, because the rates of hydration of the different chemical components of cement are different during the first hours of the process, it is not impossible that some of the cement particles may split or break during that time, depending on their chemical components.

However, in spite of the formation of the colloidal hydrates and the changes in the cement grain size, the penetration of a cement or clay-cement grout into the voids of a soil foundation is still governed for all practical purposes by the size of its largest elements, namely the hydrate covered grains of cement. This, incidentally, explains why the groutability ratio (D_{15} soil/D_{85} grout) for cement grouts is found to be somewhat higher than the same ratio for clay grouts. The gradual formation of hydrates of colloidal size also explains why, for an ordinary cement grout, a longer mixing time improves both its stability and capacity to carry sand.

CEMENT AND CLAY GROUTING OF FOUNDATIONS: THE USE OF ADMIXTURES IN CEMENT GROUTS[a]

Closure by Alexander Klein and Milos Polivka

ALEXANDER KLEIN,[1] M. ASCE and MILOS POLIVKA,[2] A. M. ASCE.—The authors appreciate the excellent discussion of their paper by Bruce E. Clark. Mr. Clark's comments are well taken with respect to the use or misuse, as he puts it, of regular rheological terminology, as applied to "practical" grouts described in the paper. Unfortunately, since no inherent terminology has yet been developed for such practical grouts, the authors borrowed from the nearest technical cousin in this field.

It was not the authors intention to consider consistency and fluidity as measures of different properties of grouts, but only as different measures of any measurable property in the evaluation of grouts. While consistency and fluidity are reciprocals in rheological terminology they are not necessarily reciprocal in practical grouts. As Mr. Clark himself points out in his discussion there is but little correlation between consistency and fluidity as measured by flow cone as observed in experiments conducted at the Waterways Experiment Station. On the other hand in tests conducted at the University of California Engineering Materials Laboratory with many materials, it appears that in some grouts considerably better correlation is obtained between these two measurements than in other grouts. It is obvious that to some degree, as the consistency measure increases (degree of torque), the fluidity measure as used by the authors (time of efflux through the flow cone) must to some degree decrease. While it is agreed that the specific gravity of the grout influences the flow cone results, the consistency measure itself will be influenced by the specific gravity of the grout, although to an unknown degree in as much as segregation also has a considerable effect upon the consistency measurements. Unfortunately, perhaps due to practical difficulties, the scientific rheologists have not shown as much interest in this field as would appear desirable. It is hoped that Mr. Clark's discussion may stimulate a greater interest by the rheologist in the practical field of grouting.

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a. Proc. Paper 1547, February, 1958, by Alexander Klein and Milos Polivka.

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CEMENT AND CLAY GROUTING OF FOUNDATIONS: SUGGESTED SPECIFICATIONS FOR PRESSURE GROUTING[a]

Closure by Judson P. Elston

JUDSON P. ELSTON,[1] M. ASCE.—The writer wishes to thank Mr. Clark and Mr. Schmidt for their interesting and informative discussions. One's only regret is that other equally well known authorities in the field did not have the time to further enlarge and contribute to this specialized but highly important phase of engineering.

This paper follows a pattern somewhat associated with the writer's past experiences while working for the United States Bureau of Reclamation at Grand Coulee, Central Valley, and other projects throughout the west. Members of the Task Committee on Cement Grouting, were asked to review and make contributions to the paper. Among these were: Edward B. Burwell, Jr., at that time with the Corps of Engineers; W. K. Seaman, who has been long associated with the Tennessee Valley Authority; and Virgil Minear (whose name through oversight did not appear in The Acknowledgement), Consulting Engineer and one of the deans of modern grouting practice, all of whose comments during the preparation of the paper were invaluable.

Packer grouting: Mr. Clark is referred to page 1548-12, paragraphs 15, 16 and 17 which would appear to cover most conditions or provisions for the use of packers.

With reference to Mr. Clark's comment on titling of schedule items for drilling grout holes, it is the writer's opinion that the use of a plug bit (normally used for grout holes), or the use of a core bit should be optional with the contractor regardless of the size of the hole. The writer has experienced granites in which plug bits were impractical in grout holes. The type of hole required, size, depth and purpose are described in the explanatory paragraphs and the statements on payment refer directly to the schedule item for the type of hole required.

The justification for a larger sized hole, that is larger than EX, is indeed at times hard to arrive at. In some types of sedimentary rocks, (such as certain limestones, sandstones, shales and dolomites) with beds of low inclination the writer has had the feeling that the slightly larger diameter hole increased the area of hole opening or crossing of seams, joints and bedding thereby permitting larger quantities of grout at thicker mixes to be injected.

It is entirely possible to force neat cement grout through 1/2-inch pipe, granted of course that all piping, fittings and valves are of equal diameter.

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a. Proc. Paper 1548, February, 1958, by Judson P. Elston.

1. Foundation Treatment—Uhl, Hall & Rich, Engrs. on the Niagara Power Project for the Power Authority of the State of New York, Niagara Falls, N. Y.

The writer is familiar with the results of the recent work done by the Corps at the Waterways Experiment Station and with similar tests conducted some years ago by the Reclamation Laboratories in Denver, Colorado. Mr. Clark will probably agree that it tends to become tedious, impracticable and at times defeat the desired end result to pump over long periods of time extremely thick mixes through hundreds of feet of 1/2-inch piping. Stoppages in pumping, line breaks, careless operation, and other normal hazards of field operation are made even more painful and costly when using small diameter piping. Contractors experienced in this type of work have in recent years come to the realization that the 1-1/2-inch diameter size for supply and return lines is the most economical and trouble free.

Mr. Clark has made an excellent point in bringing out the economic merits of wet screening over dry screening. Wet screening, however, requires a shaker mechanism and constant manual cleaning of screens. Belts, motors, bearings, guides, etc. are continually subjected to the cement grout itself resulting in shorter life and considerable maintenance. Possibly the answer to the type of screening to be required lies with the volumes and purposes for which the cement will be used.

As Mr. Clark points out, most cement mills throughout the country are now equipped with air separation facilities and are prepared to supply specially screened cement at a cost only a few cents per bag above that charged for regular grind cement.

The writer finds it difficult to agree with Mr. Schmidt ". . . that core drilling and grouting are engineering problems that can best be handled by foundation engineers working with contractors on a cost plus or upset hourly rate basis." One's own opinion based on both cost plus and force account versus contract work in government agencies, municipalities and private work indicates a trend toward increased costs and completion times over contract work with no material improvement in the quality of the completed job. The incentive and initiative of laborers, operators and foremen is always in danger of being dulled. The layout and direction of the mechanics of drilling and grouting may rightly rest with the experienced, capable worker or they may be usurped by the young or inexperienced or uncaring inspector or timekeeper in charge. There is always the temptation present to "spread" the work out, use two men to do one man's job and keep men on the payroll waiting for the next job to materialize.

The writer agrees wholeheartedly with Mr. Schmidt that the foundation for any structure does and must remain first, last and always with the owner—and with and as the responsibility of the owner's designated engineer. The writer fails to see, however, what connection this principle has to the subject pro or con of contract work. It is hard to visualize unsurmountable difficulties in preparing specifications and administering contracts on drilling and grouting work regardless of the words or pages contained therein provided they are carefully prepared. Complete control over the work to secure the desired end result is easily achieved.

In the end analysis if, as Mr. Schmidt says, successful grouting jobs require somewhat trial and error methods and improvisation, is not the burden and the challenge and the responsibility still on the individual foundation engineer? Assuming the engineer confronted with any foundation problem can assess the merits, backgrounds, capabilities, economic status, and reliabilities of the contractors and their key personnel and can select the men he will

need to assist him in carrying out the supervision of program, control would appear to be assured.

Specifications should never be made cumbersome. A guide specification used in the same manner as a glossary of words would, obviously, not be used in its entirely every time a new structure is built any more so than when a new book is printed. Specifications for a structure the size of Glen Canyon Dam on the Colorado River, for example, would require comprehensive treatment. The writer has prepared and used adequate specifications which gave full control over the work to be done in the case of drilling and grouting contracts on small earth structures and concrete spillways, pump plants and powerhouses, where the actual written material consisted of only a fraction of the material contained in the guide specifications. These guide specifications were intended to cover a number of contract problems which could arise on foundations for different types of structures. Any one specification for a particular structure foundation such as a small earth dam or ponding dike would obviously be built around and consist of only a part of the total material available in the guide specifications.

In conclusion the writer wishes again to acknowledge his indebtedness and gratitude to a few of those engineers who helped make Paper #1548 a reality, namely; Prof. Raymond E. Davis, A. Warren Simonds, Edward B. Burwell, Jr., W. K. Seaman, and Fred H. Lippold.

CEMENT AND CLAY GROUTING OF FOUNDATIONS: EXPERIENCE OF TVA WITH CLAY-CEMENT AND RELATED GROUTS[a]

Closure by George K. Leonard and Leland F. Grant

GEORGE K. LEONARD,[1] M. ASCE and LELAND F. GRANT.[2]—Mr. Schmidt's discussion of our paper is timely and his constructive criticism is greatly appreciated by the authors. In the last paragraph of his discussion, he mentioned that it would be desirable to have a tabular summary made of the projects discussed in the original paper. The results of this tabulation are given in Table I.

In this table it can be seen that there is some similarity in the grout takes per foot of hole for projects which are similar in size and have similar geologic environments. There are a number of variables which can be detected from the table. After some adjustments that are made for purely local conditions, the following features are apparent:

(1) The grout take per foot of drill hole increases as the depth of weathering in a particular grout curtain increases;
(2) For similar geologic formations, the grout take per foot of hole will be approximately equal if the maximum depths of weathering in the rocks are about the same;
(3) There is always a diminishing amount of grout taken per foot of hole in the later grouting stages. This drop is very sharp as the formation being grouted reaches its final grouted condition. For the clay-cement grouting, experience has shown when the grout take drops below .5 of a cubic foot per foot of hole that further grouting will not help materially in sealing cavities.

As Mr. Schmidt has pointed out in his discussion, this type of datum is very useful in preparing estimates for future work. It is also of very much interest to note that the trend is so consistent. It would be desirable if time were available to have all grouting jobs tabulated in this manner so that the relationship between grout takes, geologic formation, and depth of weathering could be estimated.

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a. Proc. Paper 1552, February, 1958, by George K. Leonard and Leland F. Grant.
1. Chf. Engr., TVA, Knoxville, Tenn.
2. Eng. Geologist, Schmidt Eng. Co., Inc., Chattanooga, Tenn.

TABLE
DRILLING AND GR ING SUM
SELECTED TV ROJECTS

	No. Holes	Drilling, Lin. Ft. Over-burden	Rock	Total	Gr ing, Cu Contact and Over-burden	Rock
South Holston Saddle Dam						
Curtain Grouting						
Primary	103	3,359	13,298	16,657	65,897	889,9
Secondary	105	3,420	9,812	13,232	21,737	439,7
Third Stage	109	3,681	6,435	10,116	21,672	128,4
Fourth Stage	55	2,237	4,196	6,433	7,350	109,9
Fifth Stage	5	255	666	921	84	2
Total *	377	12,952	34,407	47,359	116,740	1,568,3
South Chickamauga Dam						
Consolidation Grouting						
Primary	167	9,328	5,712	15,040	182,441	200,7
Secondary	104	5,592	2,785	8,377	25,587	79,0
Third Stage	27	1,849	667	2,516	11,770	13,1
Total *	298	16,769	9,164	25,933	219,798	292,9
North Chickamauga Dam						
Consolidation Grouting						
Primary	43	2,812	1,778	4,590	32,033	61,9
Secondary	47	3,222	1,795	5,017	21,209	26,4
Third Stage	24	1,618	812	2,439	7,834	8,1
Total *	114	7,652	4,385	12,046	61,076	96,5
Boone						
Rim Grouting						
First Stage	54	782	6,530	7,312	4,374	310,3
Second Stage	45	565	4,968	5,533	8,713	85,0
Third Stage	70	1,226	6,410	7,636	3,906	87,0
Fourth Stage	80	1,524	7,402	8,927	2,027	78,6
Fifth Stage	64	1,558	4,955	6,513	357	19,0
Sixth Stage	28	748	2,147	2,895	343	9,2
Total *	341	6,403	32,412	38,816	19,720	589,3

* Figures on this line in the six right-hand columns are not totals bu re weig

TING SUMMARY
PROJECTS

ing, Cu. Ft.

Rock	Total	Grout Per Ft. of Hole, Cu. Ft. Over-burden	Rock	Totals	Grout Per Hole, Cu. Ft. Over-burden	Rock	Totals
889,946	955,843	19.6	66.9	57.4	640	8,640	9,280
439,795	461,532	6.4	44.8	34.9	207	4,189	4,396
128,408	150,080	5.9	20.0	14.8	199	1,178	1,377
109,967	117,317	3.3	26.2	18.2	134	1,999	2,133
245	329	.3	.4	.4	17	49	66
1,568,361	1,685,101	9.0	45.6	35.6	310	4,160	4,470
200,787	383,228	19.6	35.2	25.5	1,092	1,202	2,295
79,001	104,588	4.6	28.4	12.5	246	760	1,006
13,167	24,937	6.4	19.7	9.9	436	488	924
292,955	512,753	13.1	32.0	19.8	738	983	1,721
61,939	93,972	11.4	34.8	20.5	745	1,440	2,185
26,471	47,680	6.6	14.7	9.5	451	563	1,015
8,150	15,984	4.8	10.0	6.6	326	340	666
96,560	157,636	8.0	22.0	13.1	536	847	1,383
310,315	314,689	5.6	47.5	43.0	81	5,747	5,828
85,068	93,781	15.4	17.1	17.0	194	1,890	2,084
87,054	90,960	3.2	13.6	12.0	56	1,244	1,299
78,677	80,704	1.3	10.6	9.0	25	983	1,009
19,037	19,394	.2	3.8	3.0	6	298	303
9,226	9,569	.5	4.3	3.3	12	330	342
589,377	609,097	3.1	18.2	15.7	58	1,728	1,786

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INCREASED RESISTANCE TO DEFORMATION OF CLAY CAUSED BY REPEATED LOADING[a]

Closure by H. B. Seed, R. L. McNeill and J. DeGuenin

H. B. SEED,[1] A. M. ASCE, R. L. McNEILL,[2] J. M. ASCE and J. DeGUENIN.[3]—The authors appreciate the constructive comments of Messrs. Ahlvin and Trollope with regard to the application of the effects described in the paper to actual pavement performance. The data reported by Mr. Ahlvin in Fig. 1 clearly indicate that stress repetitions on a single wheel path induced a stiffening of the soil similar to that reported in the paper; however, stress repetitions on adjacent wheel paths apparently destroyed the cause of the stiffening. The authors agree that this effect requires careful consideration in extrapolating the results of laboratory triaxial compression tests to actual field conditions. They are inclined to believe that the permanence of the stiffening effect under these conditions may well depend largely on the magnitude of the deformations induced by the wheel loads. When the deformations are large the stiffening effect will be destroyed (in a manner similar to that shown in Fig. 14) but if the deformations resulting from wheel load applications in adjacent paths are small, the stiffening effect may well remain.

Similar considerations would apply to the test data reported by Mr. Ahlvin in Fig. 2. Here again the increases in stress (between 30 and 50 per cent) may have been so great that any stiffening effect induced by the initial stress repetitions was destroyed (as illustrated in Fig. 14 of the paper). However, it is also doubtful whether the 2,000 repetitions of the lower wheel loads would induce any appreciable stiffening of the soil in these tests; in the experience of the authors 10,000 or more stress repetitions are usually required before any appreciable stiffening effect is produced.

Dr. Trollope's comments on the influence of the tire width on the depth of soil in which an increased resistance to deformation might be developed illustrate another factor requiring careful consideration in assessing the influence of laboratory test results in actual pavement performance.

The great complexity of any analysis of pavement behavior is no doubt responsible for the largely empirical methods of design currently in use, and even if suitable analytical procedures cannot be developed in the foreseeable future, clarification and discussion of all of the factors involved will undoubtedly lead to better understanding and to improvements in design methods.

a. Proc. Paper 1645, May, 1958, by H. B. Seed, R. L. McNeill and J. DeGuenin.

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3. Formerly Graduate Research Engr., Inst. of Transportation and Traffic Eng., Univ. of California, Berkeley, Calif.

The contributions of Messrs. Ahlvin and Trollope in this regard are a valuable supplement to the original paper.

GEOLOGICAL FACTORS IN TUNNEL CONSTRUCTION: TOOLS AND TECHNIQUES[a]

Closure by A. B. Cleaves

A. B. CLEAVES,[1] M. ASCE.—The comments by Mr. A. A. Eremin[2] pertinent to the paper are appreciated. However, one could expand on "tools and techniques" considerably.

Because the original paper was written some years prior to publication, and because the problems of residual stresses in the rocks of the earth's crust and the effect of these stresses on underground excavations have come to be of greater importance than ever before, some discussion of their determination arrears warranted.

During the past decade more and more attention has been focused on problems of rock mechanics, involving the elastic properties of rocks and rebound phenomena. The ROCK MECHANICS SYMPOSIUM given under the auspices of a joint committee of the ASCE and the Geological Society of America on November 6, 1958 in St. Louis emphasized the significance and importance of these problems.

Because residual, tectonic and other stresses in excess of normal overburden stresses are so common in many parts of the world, and because they have a dynamic bearing on both the design and actual construction of underground projects they should be investigated, in part prior to and also during excavation. Their study involves "tools and techniques" not touched upon in the original paper.

A study of the elastic properties of rock and foreknowledge of the "trend" of magnitude of the forces involved can greatly aid the designers, especially with reference to the vaulting of the roof, the payline of the chamber or tunnel walls, the scope, spacing and length of rock bolts and/or timber supports, and the thickness of a lining.

Both laboratory and field studies may be used, and both supplement and serve as checks on each other. Two-dimensional photoelastic analyses of rock cores may be made in the laboratory and these may give a reasonably accurate representation of the possible stress pattern.(1) Laboratory investigations, however, have the disadvantage that the results may not accurately represent the in-place properties of the over-all rock mass. In part, and because of this a field seismic (geophysical) method can be used as a check. The velocity of the artificially created seismic waves can be accurately measured. The product of the square of the velocity with the mass density is roughly the modulus of the elasticity. As a further check, in-place determinations may be made of the rock properties using spherical or cylindrical

a. Proc. Paper 1648, May, 1958, by A. B. Cleaves.
1. Prof. of Geology, Washington Univ., St. Louis, Mo.
2. Proc. Paper 1881, p. 9, Dec. 1958.

shells under internal pressure while imbedded in the rock at the site.[2] In addition to these techniques plate bearing tests may be performed on the rock at the site.

If these preliminary studies show the presence of significant inherited stresses, additional techniques can be employed during excavation to delineate in detail the direction of, and extent of the forces to be coped with. These may involve the flat-jack method described by M. E. Tincelin;[3] Huggenberger deformeter points fixed in reinforced concrete ribs (if used); sensitive clinometers to detect any rotational movements of blocks; stress and strain meters; and precise extensometer measuring tapes.

Practical usage of many of these techniques is described in papers involving recent work in the Snowy Mountains in Australia,[4] and work of the Hydroelectric Power Commission of Ontario in the Niagara area.[5]

Because of increasing interest in and actual underground construction throughout the world, more attention is being given to the problems involved and the research efforts in progress promise to result in more and better "tools and techniques".

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GEOLOGICAL FACTORS IN TUNNEL CONSTRUCTION: GEOLOGY AND TUNNEL DESIGN[a]

Closure by A. B. Reeves

A. B. REEVES,[1] M. ASCE.—The examples, cited by Mr. Eremin, of encountering unexpected conditions in tunnel construction are well known to the writer, and point out the hazards that may be created by lack of sufficient preliminary investigations.

A more recent illustration of this point may be indicated by the two tunnels now under construction through the Continental Divide west of Denver, Colorado, by the Denver Water Board. The Vasquez Tunnel, three and one-half miles long, was constructed through an area in which very little detailed geologic study had been made, though the general geology of the region was well known. The failure to anticipate the requirement for supports in the tunnel, in amount as well as strength, has resulted in an increase in cost over the original contract price of thirty-five per cent.

Compared to this case, the Harold D. Roberts Tunnel, twenty-three and one-fourth miles in length, through a highly mineralized area, was given a thorough geologic study. While the excavation of this tunnel is only two-thirds completed, the pattern of the rock structure so far encountered has been found to agree very closely with the geological forecasts.

Mr. Eremin's reference to the use of the equations of Rankine and Boussinesque is well taken, both of which are well recognized and widely used. However, the availability of adequate geologic interpretation of the area through which a tunnel is proposed may well determine, in advance, the necessity for the application, as well as the limitations of these equations.

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a. Proc. Paper 1649, May, 1958, by A. B. Reeves.

1. Chf., Irrigation Operations Div., Bureau of Reclamation, Denver, Colo.

DESIGN AND PERFORMANCE OF VERMILION DAM, CALIFORNIA[a]

Discussions by R. W. Spencer, P. J. West, J. H. Birman and B. R. Laverty

R. W. SPENCER,[1] M. ASCE.—The authors are to be congratulated on their excellent paper covering the performance as well as the design of the Vermilion Dam. The construction and successful operation of this dam is a fine example of the very necessary cooperation between Expert Consultants, Engineer-Contractors and the Owner's own engineering staff in solving the problem of getting a safe and economical dam on a very difficult foundation.

For the purpose of this discussion we will consider the Consultants or Consulting Board only. From the Owner's point of view the selection of a Consulting Board should be influenced by the following requirements.

1. The Board should have specialists in several lines, for example, foundations, materials to be used in the structure, and geology.
2. Board Members should be selected who will work together harmoniously without submerging their individual judgments.
3. The Board Members should be available throughout the job and not just for preliminary consultation.
4. The Board Members should have a sense of economy.
5. Above all, the Board Members should be able to give mature judgment in their respective specialties.

Occasionally, owners retain a Consultant or Consulting Board merely to give preliminary advice or to lend their reputation to a project. Thereafter the Consultants have little or no contact with it. When this occurs, Consultants have found from sad experience that failure by others to adjust designs to fit the varrying conditions that nature provides on the job may result in serious errors. Consequently, Consultants are apt to take the position that their recommendations must be ultra conservative so that even a careless designer or builder will achieve a safe structure. Such a structure is rarely economical and if the site conditions and construction practices wander far enough from the original assumptions, it may not be adequately safe.

Dr. Terzaghi, the senior author of the paper, has covered the important subject of the relationship of "Consultants, Clients and Contractors" very well in an article published last year in the Journal of the Boston Society of Engineers. This article merits reading by every engineer dealing with important structures where safety of the public is involved.

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a. Proc. Paper 1728, August, 1958, by K. Terzaghi and T. M. Leps.

1. Mgr., Eng. Dept., Southern California Edison Co., Los Angeles, Calif.

In the case of the Vermilion Dam, the Consultants, the Engineer-Contractor and the Owner's staff worked diligently throughout the job to fit the structure to the highly-varying conditions provided by nature. Many of these conditions could not be determined in advance by any reasonable amount of exploration. The excellent performance of the Vermilion Dam is convincing evidence that a tough problem has been well solved.

P. J. WEST.[1]—The conclusions reached in this informative paper may well serve as practical guides to engineers and geologists charged with the design of major earthfill dams to be located on thick, erratic, alpine glacial deposits where (1) complete and accurate interpretations of subsurface information based on pre-construction investigations are virtually impossible to make, and (2) behavior of underseepage pattern and flow cannot be either fully ascertained or understood until after careful observation of the performance of the dam during the filling of the reservoir.

Of especial interest to the writer in regard to these items, was his association with the development of the local subsurface glacial geology both before and during the construction of the dam, and with later attempts to account for occurrences of unusual underseepage phenomena. Accordingly, it is believed that a brief discussion of events and studies which led to rather significant conclusions regarding the causes of certain underseepage abnormalities observed during the initial partial filling of the reservoir, will be of additional interest.

The authors, in their discussion of Seepage Losses, pointed out that toe drain discharge Q. in excess of the amount that could normally be anticipated from reservoir underseepage alone, could be accounted for by a temporary increase of the quantity of water entering the ground adjacent to the toe drain and beyond the area occupied by the reservoir, from heavy rainfall, or snow melt. However, prior to the concept that underseepage discharge irregularities which developed during the initial filling of the reservoir could be accounted for by meteorological events, it was considered likely that such irregularities in underseepage were attributable to changes in the average mass permeability of the subsoil resulting from internal subsurface erosion, deposition, and the subsequent flushing of water veins. Such an irregularity in the Reservoir Head-Toe Drain Discharge Curve is represented by the shaded area above the smooth roughly parabolic "ideal" H-Q curve C2, Fig. 1, which represents the simple relationship between the loss of water Q which seeps through the foundation from the reservoir and the reservoir water level H.

In the course of evaluating the physical conditions which caused a noticeable variation in toe drain discharge vs. time which occurred during the Spring of 1954 before the intake gates were closed on November 12, 1954, it became reasonably evident that ground water, flowing into the toe drain during the snow melt and runoff season could increase the toe drain discharge as much as about 0.20 cfs above a so-called "base" underseepage flow caused by normal underground movement of water downstream through the valley fill.

Since the initial major departure from the ideal H-Q curve (Fig. 1), which occurred during the period January 11 to May 30, 1955, covered the observed local snow melt and runoff season in the Vermilion Dam area, it became

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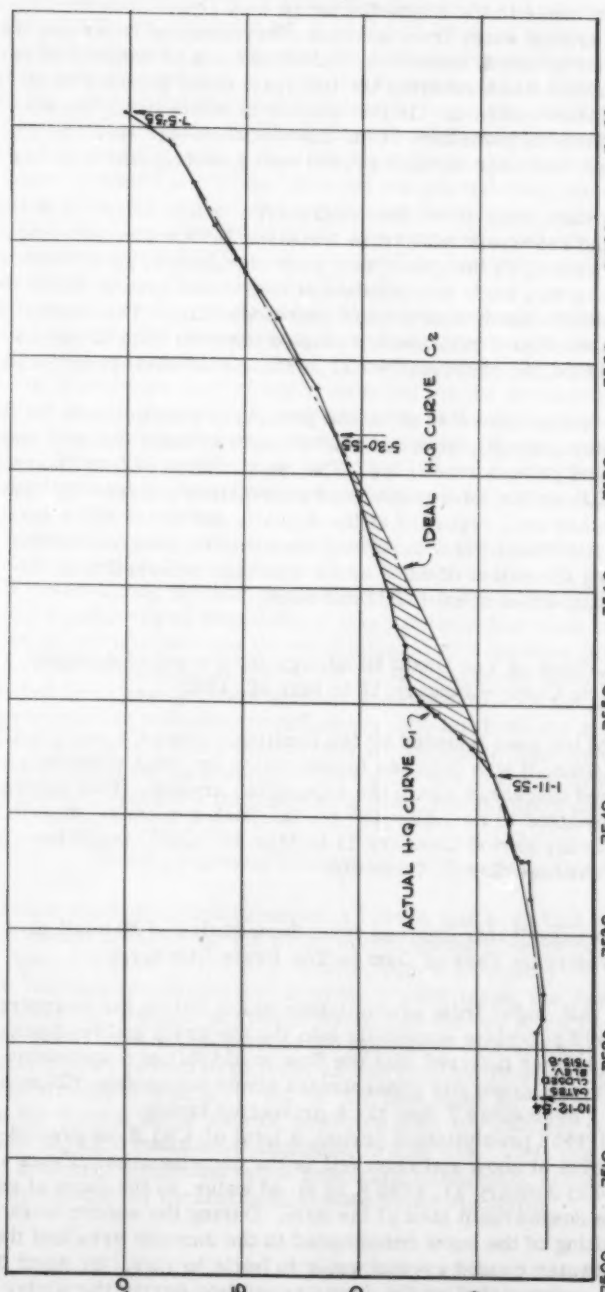

FIG. 1. RELATIONSHIP OF TOE DRAIN DISCHARGE TO RESERVOIR WATER SURFACE ELEVATION DURING INITIAL PARTIAL FILLING OF THE RESERVOIR - VERMILION DAM.

apparent that the increase in toe drain discharge could have been created by increased flows of ground water from sources other than the reservoir into the toe drain. It was believed, therefore, that in addition to seepage of reservoir water, incremental flows entering the toe drain could possibly be derived from the following other sources: (1) Percolation of meltwater from snow accumulated on the downstream face of the dam between Sta. 9+00 and Sta. 42+00 and (2) Surface and near surface ground water contributed from the abutment areas.

In analyzing the variations of toe drain discharge, which occurred during the time in which the reservoir rose from elevation 7515 in October, 1954, the month prior to the closing of the gates, to a peak elevation of about 7600 in July, 1955; reference was made to a number of charts and graphs which were prepared from available basic weather and hydrologic data. The attempt was made to determine whether a relationship existed between such things as temperature, precipitation, reservoir water surface elevation, runoff, and toe drain discharge.

A diagram was then prepared to show the periods of precipitation (in form of rain or snow water content), together with the approximate dates of snow melt, local runoff and ground water rise. The construction of this diagram was based principally on the interpretive and correlative information derived from the basic weather data recorded at the damsite and at the other local weather stations. A chronological series of construction progress aerial photographs showing the extent of snow cover was also referred to in establishing the limiting dates of snow fall and melt.

Calculated Increase of Toe Drain Discharge for Period of Anomaly in Curve—January 11 to May 30, 1955

By integration of the area bounded by the limiting curves C1 and C2, (Fig. 1), plotted against time, it was possible to determine the total volumetric quantity of increased discharge above the expectable amount. This incremental quantity was calculated to be 1.69×10^6 cu. ft. Such a quantity, distributed evenly over the 139-day period (January 11 to May 30, 1955), would have been represented by an average flow of 0.141 cfs.

Relation of Seepage Water Derived from Snow Melt and Rainfall at Downstream Face of Dam to Toe Drain Discharge

It was believed that water from precipitation which fell on the downstream face of the dam could percolate eventually into the toe drain and be discharged therefrom. It was further believed that the flow would follow a somewhat inclined path downstream, generally concentrated above the random fill material, within the highly permeable 2-foot thick protective facing.

During the 1954-1955 precipitation period, a total of 1.93 ft. of precipitation (water), in form of snow and rain, fell in the damsite area. From November 12, 1954 to January 11, 1955 8.39 in. of water, in the form of snow, accumulated on the downstream face of the dam. During the second week of January, 1955, melting of the snow commenced in the damsite area and the percolation of meltwater caused ground water to begin to rise. By April 28, all snow which had accumulated on the downstream face during the winter had

melted and also by that time approximately 13 in. of additional precipitation (water) had fallen. By May 30, a total of 1.93 ft. of precipitation (water) had fallen in the damsite area during the 1954-55 precipitation season.

It was considered that if all of the precipitation (1.93 ft of water from snow melt and rain) which fell on the entire surface area of the downstream face of the dam between Sta. 9+00 and Sta. 42+00 during the period, November 12, 1954 (start of ppt.) to May 30, 1955 (termination of local snow melt and significant precipitation) percolated into the toe drain during the snow melt period, the total amount of water entering the toe drain would amount to 1.67×10^6 cu. ft. and would represent an average discharge of 0.139 cfs. This percolation period corresponded to the time interval during which the "actual" curve C1 departed from the "ideal" curve C2, (Fig. 1).

It was, of course, questionable whether one should assume that all precipitation falling on the downstream face of the dam would ultimately be collected by the toe drain. It was believed, therefore, that if a reasonable allowance of 20% were made for loss of water entering the toe drain from precipitation sources, due to evaporation, underseepage and other causes, wherein the percolating water would not enter the toe drain, the quantity would have been reduced from 1.67×10^6 cu. ft. to 1.34×10^6 cu. ft. and the corresponding discharge flows would have been reduced from 0.139 cfs to 0.107 cfs distributed over the 139-day period.

Obviously, the incremental discharge of 0.107 cfs would not have commenced immediately on January 11 or ceased on May 30, nor would the flow at all times during the 139-day period have been constant at 0.107 cfs. However, for purposes of simplicity it was assumed that such was the case.

The difference between the total integrated discharge quantity of 1.96×10^6 cu. ft. and the quantity of 1.34×10^6 cu. ft. that could have been reasonably expected from precipitation seepage water was 0.35×10^6 cu. ft. This quantity corresponded to a discharge deficiency of about 0.03 cfs distributed over the 139-day period. It was considered that this small flow deficiency must have been accounted for by seepage entering the toe drain from sources other than from rainfall and snow melt seepage under the downstream face.

Effect of Ground Water Rise on Toe Drain Discharge

During the Spring and Summer of 1954 it was seen that a rise in ground water could increase the toe drain discharge an average of 0.10 cfs for over a period of at least 2-1/2 to 3 months, (April 10 to July 10, 1954). Accordingly, it was understandable that at some period during the January 11 to May 30, 1955 interval, seepage of ground water into the toe drain from the abutment areas, could have supplemented the discharge rate at least 0.10 cfs for a similar 75-day period to provide an incremental flow amounting to a total discharge volume of 0.65×10^6 cu. ft. On this basis the toe drain discharge flow, from the ground water source, prorated over the 139-day period, January 11 to May 30, 1955 could have amounted to 0.06 cfs. Thus the discharge deficiency of 0.03 cfs, mentioned in the preceding paragraph, could have been more than accounted for by abutment ground water discharge.

CONCLUSIONS

It appeared that a correlation existed between the anomalous toe drain discharge, which occurred during the period January 11 to May 30, 1955, and percolation into the toe drain of water originating from rainfall and snow melt on the downstream face of the dam, and with seasonal fluctuations of the abutment area ground water table. From this study it was concluded, therefore, that the irregularity in the H-Q curve (Fig. 1, Curve C-1) could be entirely accounted for by a seasonal incremental flow of water into the toe drain entirely separate from reservoir caused flow.

J. H. BIRMAN.[1]—Messrs. Terzaghi and Leps have done a remarkable job in describing so succinctly a complex project which required many years to complete. This writer wishes to express his congratulations for a well prepared report and his thanks for the privilege of contributing, in a minor way, to the project.

In view of the success of the Vermilion Dam, there are several aspects of the local geology which bare further discussion. These features may be significant in regards to planning for other dams on thick fill in glaciated canyons.

As there had been at least four separate ice advances through Vermilion Valley, four layers of glacial till might have been expected—each till layer associated with glacifluvial outwash. Since till is directly deposited by ice with little or no size sorting and outwash is deposited by meltwater streams, it appeared important to determine the amount and distribution of each type of material in order to predict gross permeability within the entire blanket of fill on the bedrock overlain by the dam. It was impossible to distinguish between the till and outwash in the drilling, and therefore, the solution to the problem had to come from the detailed history of geological events in the area. It now appears that because of stream erosion during the times between the ice advances, only the till corresponding to the last ice advance still remains in Vermilion Valley. This is the Mono till, as shown in the authors' Fig. 6a and 6b. Thus, all of the underlying materials are believed to be outwash.

In the area of the damsite the upper part of the Mono till is loose and earthy, and the matrix is somewhat sandy. Lack of much silt and clay suggests winnowing by melt water during deposition. The lower portion is very compact and has a significant content of silt. The lower portion is interpreted as basal till formed of materials carried along the base of the glacier and plastered over the area. The debris within and upon the melting ice was let down on the basal till already in place. The silt of the basal till was probably derived by local gouging of a silt stratum (Authors' p. 19) within the upper part of the outwash deposited as a down-valley derivative of the advancing glacier and later overridden by it.

The silt stratum clearly indicates that a pond or small lake existed for some time in the area where the dam has been built. The deposit is outstandingly laminated; and, although silt is predominant, there is a little clay and locally fine sand. The silt layers are gray, minutely cross-bedded internally, and ripple-marked. Most of the layers are 0.5 cm to 1.5 cm thick, and there

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is some lateral variation in thickness. Gray color is due to content of heavy, ferromagnesian minerals such as hornblende and magnetite. The clay layers are white to pale yellowish green, 1 mm to 2 mm thick. There are nowhere cross-bedded or ripplemarked, and are remarkably constant in thickness. The darker, coarser-grained layers are believed to represent the summer deposits, and the light clay layers the winter deposits of annual varve pairs. If the varve interpretation is correct, the lake existed for about 1,000 years in the time interval between the two latest ice advances in Vermilion Valley. The position of the silt stratum and lack of visible organic detritus suggests that the advancing Monoage glacier (the latest) was not far up-valley while the lake was in existence. The lake deposits and associated deposits are interpreted as proglacial outwash subsequently overridden by the glacier itself.

The question occurs as to what dammed the lake which seems to have existed so long at the site the present dam and which was developed on almost identical materials. Moraine damming or bedrock damming are easily eliminated by the abundance of geological evidence. Only outwash was (and still is) available in a topographic position to impound a body of water at the elevation of the silt stratum. This outwash was deposited from small glaciers on the south wall of the valley, downstream from the damsite, and from the San Joaquin trunk glacier which thrust a lobe into the Mono Creek reentrant (Fig. 1). It is stressed that the moraines of these glaciers were not formed until after the deposition of the outwash within which the silt stratum occurs. The outwash was formed while the glaciers were advancing and before they reached their maximum extent. Thus, it seems that an ancient geological experiment indicated that a lake of some duration could successfully be impounded in Vermilion Valley. This seemed especially obvious in that during the geological experiment the more impervious Mono till, which now underlies the damsite area of the present lake, had not yet been formed.

B. R. LAVERTY,[1] J. M. ASCE.—This paper is an instructive contribution to the literature on earth dam design. The authors' detailed discussion of the problems resolved during the design and construction of Vermilion Dam gives insight into the application of practical soil mechanics in modern dam construction. In view of the writer's association with the project both during and after the construction period, it is his belief that a fitting supplement to the paper is a discussion of the observed performance of the dam during several seasons of satisfactory operation.

To properly monitor the performance of the dam, a comprehensive observation program has been maintained. It consists primarily of performing frequent piezometer measurements, seepage measurements and settlement and alignment surveys. These measurements were begun before the dam was completed. Aerial photographs have also been utilized to define seepage patterns in the natural ground downstream from the dam.

Observation Program

During the first partial filling of the reservoir in 1955 and again during its first filling to capacity in 1956, resident observers at the damsite measured

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all piezometers and seepage weirs weekly. During the periods of high reservoir, key piezometers and weirs were read daily. This information was reported each day to the Engineering Department of the Southern California Edison Company. The performance of each well was plotted and analyzed daily to maintain a current evaluation of the pattern and amount of seepage developing within the embankment and through the complex foundation materials.

Since the dam has performed satisfactorily for four seasons, the frequency of the observations has been somewhat reduced and the period between measurements is now contingent upon the operation of the reservoir. During the spring and summer, when the reservoir is filling rapidly, the piezometers and the leakage weirs are read bi-weekly. During the winter months when the level of the water in the reservoir is low, the frequency of observations is reduced to bi-monthly measurements of the piezometers and monthly readings of the weirs. The magnitude of the observation program may be inferred from Table 1, which indicates the number of individual measurements made at each well and weir since the dam was placed in operation.

TABLE I

Number of Observations Made at Piezometers and Weirs
1955 - 1958

	1955	1956	1957	1958
Piezometers	42	204	11	7
Weirs	42	208	20	22

During 1956 the more important piezometers were read nearly every working day. The less significant piezometers were read with about one-fourth the frequency of the key wells.

Fifty-three piezometers were installed during construction of the dam. However, in areas where more complete information was subsequently found desirable, additional piezometers were installed following the construction period. At present, there are 100 piezometers maintained and read regularly.

The level of the water in the piezometers is sounded by using a Fisher M-Scope Water Level Indicator. The 3/4 inch diameter piezometers used in the shell and along the toe of the dam have proven adequate in size. Unfortunately, the deep foundation piezometers which were 3/8 inch plastic tubing have proven less successful because of their small size.

Analysis of Observation Data

Piezometers

During 1955, the first season of operation, the reservoir was partially filled. The maximum pore water pressure anticipated at each piezometer at full reservoir was predicted by extrapolating the curves plotted from the piezometric data obtained under these partial loaded conditions. The stability analyses of the critical sections of the dam were repeated utilizing the values of the predicted maximum pore water pressures. The analyses indicated that, as finally constructed, the dam would be amply safe under the seepage regimen developed by full hydrostatic loading.

After each observation during the reservoir filling period, the measured piezometric head in each of the wells was compared with trend lines terminating at the maximum predicted values developed for the final stability studies. When steady state seepage was developed under full reservoir conditions, it was found that the predicted maxima were conservative and that the actual piezometric heads developed were less than anticipated. Table II of water surface elevations in representative key piezometer wells shows the predicted pressures and the maximum measured pressures at full reservoir. During 1955 the reservoir reached a maximum elevation of 7603. The pressures are expressed in elevation of piezometric water surface.

TABLE II

Well *	Design Final W.S. Elev.	Maximum Actual W.S. Elevations 1955	1956	1957	1958
P-25	7564	7545.1	7553.1	7548.1	7547.1
P-26	7553	7534.3	7543.0	7536.0	7534.0
P-29	7526	2514.5	7520.0	7516.0	7515.0
P-9	7500	7491.7	7495.0	7495.2	7494.6
P-101A	7553	7536.8	7545.0	7539.0	7537.9
P-101B	7545	7532.3	7541.0	7535.0	7534.0
P-102A	7514	7507.5	7512.0	7509.0	7505.7
P-102B	7517	7512.7	7518.0	7517.0	7512.5

As shown in Table II, the highest observed piezometric levels occurred during the first filling of the reservoir in 1956. Slightly decreased pressures were measured in succeeding years. This decrease is attributed to the development of natural drainage patterns and seepage paths through and beneath the dam. The high piezometric levels indicated in 1956 are also in part attributed to the exceptional precipitation that occurred during this season. Studies have shown that heavy precipitation or rapid snow melt significantly influence the performance measurements.

The data obtained from the observation wells are tabulated and plotted to provide a continuous record of the performance of each well. The analysis of these plotted data is done promptly to insure the early detection of any deviation from previous patterns.

Leakage

As a supplement to the authors' extensive studies of seepage through the heterogeneous glacial deposits comprising the foundations of this dam, the late Prof. Donald W. Taylor constructed, during the design period, an electrical analogy model of the embankment and foundation. From the results of his studies, Professor Taylor estimated the total amount of seepage at full reservoir would be on the order of 4 cfs. His estimate compares remarkably well with the measured maximum seepage of 4-1/2 cfs which is the maximum measured discharge through the toe drain.

In a manner similar to that for the piezometer records, individual continuous plots are also kept of the measured discharges through the seepage weirs.

*The locations of these wells are in cross sections shown on Fig. 6-a, Proc. Paper 1728.

Fig. I illustrates the relationship of measured seepage to reservoir elevation. Shown on this figure are the quantities of seepage collected from the toe drain discharge lines and the seepage collected downstream from the dam.

Of particular interest, and demonstrated in this figure, is the remarkable sensitivity of seepage discharge to reservoir elevation. The nearly immediate response of seepage to a change of reservoir elevation indicates the development of mature seepage paths through the embankment and foundation. This immediate response together with the stable piezometer performance indicates that no latent high seepage pressures are developing which could cause piping or otherwise reduce the stability of the embankment.

Since heavy precipitation and rapid snow melt influence performance measurements, it is necessary to evaluate the significance of erratic peaks or anomalies in otherwise consistent curves. Therefore, continuous records are maintained of precipitation and temperature at the damsite. An example of the application of such weather data was the investigation to explain the reason for the toe drain discharge increase in April, 1958, during a lowering reservoir elevation as shown in Fig. I. The answer was found by examination of the daytime temperature and the precipitation records for the month.

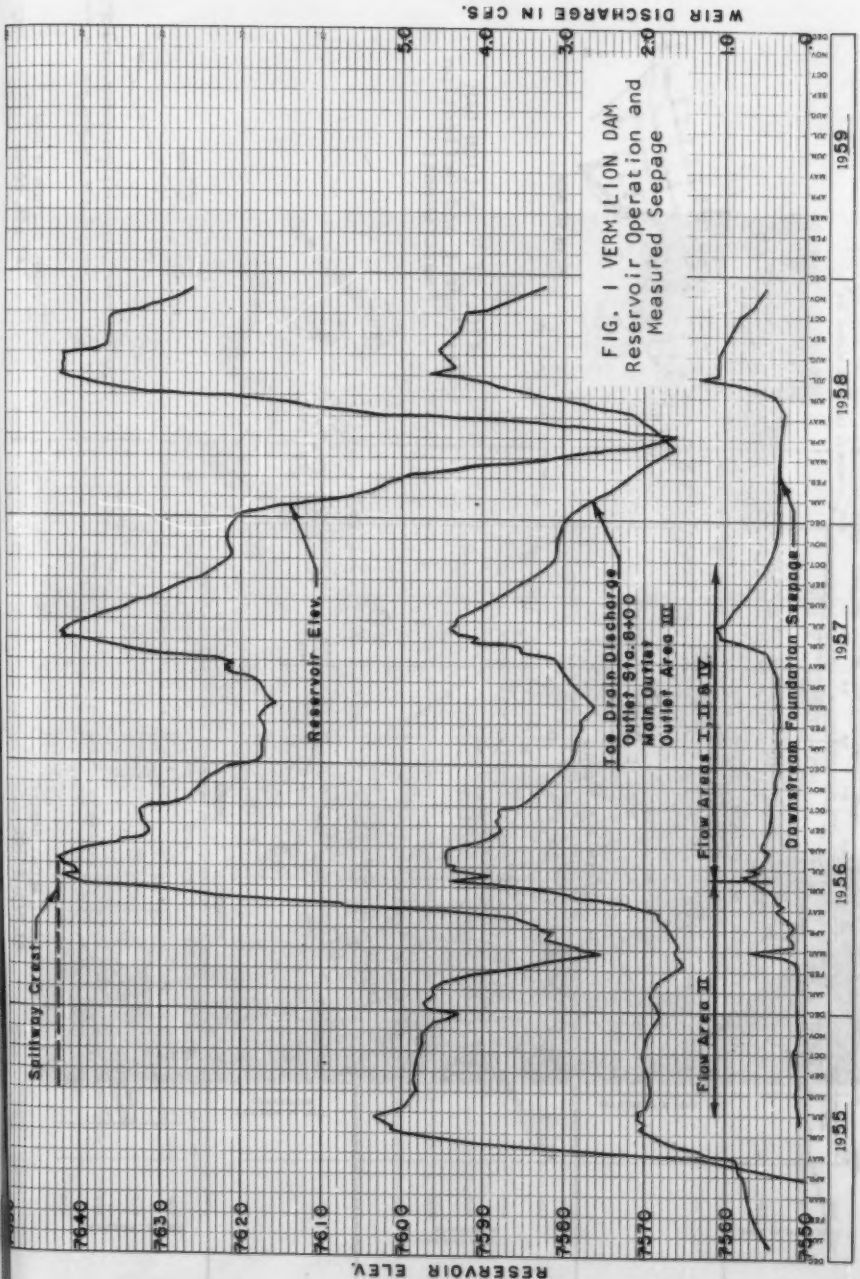
In April, 1958, the area immediately downstream from the dam, as well as the downstream slope of the embankment, was covered by several feet of snow. As the daytime temperature rose above freezing, the local snow melted rapidly causing a general increase in ground water elevation. Some of this water undoubtedly entered the toe drain, and additional water also seeped through the embankment into the drain. This same rise in temperature marked the beginning of the spring run-off period at higher elevations in the watershed. Consequently, shortly thereafter the gates were closed and the reservoir began to fill.

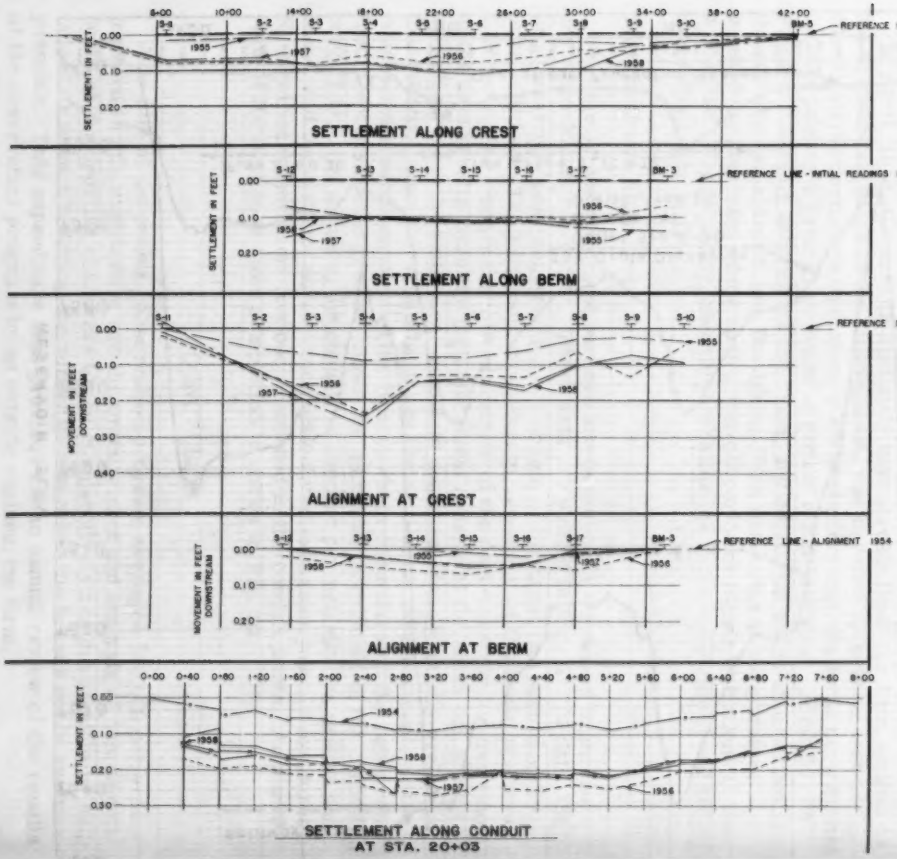
Settlement and Alignment

Settlement and alignment surveys are periodically conducted to measure any embankment movement. These data are plotted in Fig. II. To date, the total maximum post-construction settlement of the crest has been slightly over 0.10 foot. The maximum downstream movement at the crest has been 0.27 foot. The remarkably low magnitude of the post-construction movement of the dam results from the high placement density of the earth embankment as well as from the settlement-resistant qualities of the glacially preloaded foundations. Also shown on Fig. II are the settlement measurements at the concrete discharge conduit. This figure, which demonstrates the settlement from the constructed grades, shows the maximum settlement of the conduit to be 0.27 foot. In addition, the conduit is checked periodically for cracks and leaks, and the distance across each articulated expansion joint is measured. No significant movement has been recorded in the conduit joints. An aerial view of the dam soon after completion is shown in Fig. III.

CONCLUSION

The cost of the present observation program averages $10,000 per year. Included in this expenditure are the procurement and analysis of data and the costs of transporting observers to the comparatively remote damsite by helicopter or snow-cat during the winter months when the access roads are impassable. This expenditure also provides for an annual review of the results of the observation program by an outside engineering firm.


WEIR DISCHARGE IN CFS.
5.0
4.0
3.0
2.0
1.0
.0
FIG. 1 VERMILION DAM
Reservoir Operation and
Measured Seepage
Ice Drain Discharge
Outlet Sta. 8+00
Main Outlet
Outlet Area III
Flow Areas I, II & IV
Flow Area II
Downstream Foundation Seepage
Reservoir Elev.
Spillway Crest
RESERVOIR ELEV.
7640
7630
7620
7610
7600
7590
7580
7570
7560
7550
1955
1956
1957
1958
1959


SETTLEMENT ALONG CREST
REFERENCE LINE - INTIAL READINGS 1954
SETTLEMENT ALONG BERM
REFERENCE LINE - INTIAL READINGS 1954
ALIGNMENT AT CREST
REFERENCE LINE - ALIGNMENT 1954
ALIGNMENT AT BERM
REFERENCE LINE - ALIGNMENT 1954
SETTLEMENT ALONG CONDUIT
AT STA. 20+03

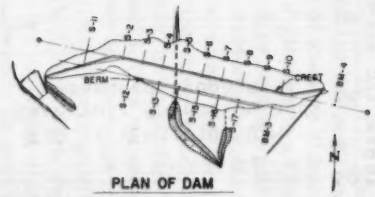
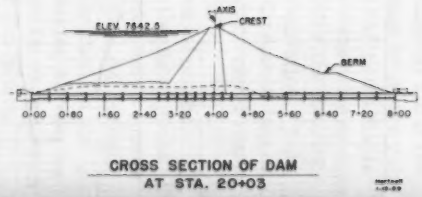


FIG. 11 VERMILION DAM
Settlement and Deflection
Measurements



SETTLEMENT ALONG CONDUIT
AT STA. 20+03

CROSS SECTION OF DAM
AT STA. 20+03

VERMILION DAM

(photographed soon after completion)

FIG. 111

Dr. Terzaghi has reviewed the performance data periodically. As he has stated in technical papers and in talks before professional groups, the men associated with the collection and evaluation of data of this type must not be satisfied with the compilation of volumes of statistics. Rather, he has urged them to analyze critically and to diligently interpret the significance of the measurements. His penetrating insight into the significance of observed data has been a great aid in forecasting the performance of Vermilion Dam. His analytical techniques have indeed offered a challenge to those associated with him on this project.

T. M. Leps, who coordinated and supervised the technical design of the dam for Southern California Edison Company, has likewise supervised all post-construction observation programs and performance studies. The observation program is considered an essential part of the proper operation of this dam of the Big Creek-San Joaquin Hydroelectric Development. The careful and painstaking performance of the program by such Edison personnel as R. N. Gomolak and D. C. Willer, J. M. ASCE, is absolutely essential to its success. Further, the information and understanding gained regarding the complex patterns of seepage through and beneath the dam add to the available knowledge for application in the design of future projects.

The observation program, including the surveys and visual inspections of the dam, has thus enabled the owner and the Department of Water Resources of the State of California, the public agency accountable for the safety of the dam, to evaluate adequately the performance of the dam.

PROCEDURE FOR RAPID CONSOLIDATION TEST[a]

Discussion by Carl W. Fenske

CARL W. FENSKE,[1] A. M. ASCE.—There are many occasions in soils work when the time required for a conventional consolidation test is a disadvantage. The author has presented a novel and intriguing approach for shortening the time required to obtain consolidation test data. However, after analyzing data from a number of standard consolidation tests on natural soils, and after performing a limited number of tests by the author's rapid procedure, the writer has some comments on and several questions about the rapid consolidation test procedure.

An analysis was made of data from twenty-two consolidation tests from the files of the writer's firm; the test specimens were 2.5-in. in diameter and 0.5 in. thick. These tests were made on (1) very soft, normally-consolidated clays; (2) stiff to very stiff, overconsolidated clays; and (3) very stiff to hard clays from such depths (127 ft to 1512 ft) that they could be classified as normally-consolidated. From a study of ninety-three time curves, the average slope for the steepest tangest was 0.51 for the normally-consolidated clays and 0.56 for the overconsolidated clays. It was also evident that this slope was not constant but generally increased with increasing load on the sample. The difference between the slopes of the steepest tangent observed in our laboratory presented here and that given by the author does not necessarily invalidate the author's rapid-test procedure, but it does indicate that the selection of the steepest-tangent slope to be used with the rapid procedure would be made with care in order that the results are comparable with the results from a standard consolidation test.

The author presents results of a test on a prepared sample of bentonite. Good conformity was shown for the one dial reading-log time curve given. Has the author always obtained this good conformity for tests on natural soils? A void ratio-log pressure curve for the bentonite sample computed from rapid test results was given but no comparison was made with results from a standard test. Can the author present some information on observed comparisons between results from tests on natural soils using the rapid and standard test procedures? The range of any discrepancies he may have encountered will be particularly informative.

Our analyses indicated one other factor of significance that was particularly evident from tests, including a few using the rapid procedure, on the stiff, overconsolidated clays common in the coastal plains of Texas. For loads that are within the recompression portion of consolidation curve, the dial reading-log time curve frequently has a somewhat nebulous relation to a theoretical

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a. Proc. Paper 1729, August, 1958, by Hsuan-Loh Su.

1. Sr. Soils Engr., McClelland Engineers, Inc., Houston, Tex.

curve. For these cases, analysis of a complete 24-hr curve by conventional curve-fitting procedures is difficult, and analysis of a portion of the curve, as for the rapid procedure, is extremely difficult. The author's comments on any experience he has had with this situation would be of interest.

One other point which the writer believes is of consequence has nothing to do with the theory or mechanics of the rapid procedure, but instead is more of a matter of economics. In the laboratory of the writer's firm, one man usually handles six to eight consolidation tests simultaneously using the conventional test procedure. This assignment requires a maximum of one-half of his working day, leaving him free to perform other tests during the remainder of the day. Each of our usual seven-load-increment consolidation tests, therefore, requires about 0.5 man-days. On the other hand, a test by the author's quick procedure can be performed in a single day but requires the full time of a technician to accomplish it. Furthermore, our limited experience with the rapid procedure indicates that the man must be quite experienced to perform the test alone. A consolidation test by the rapid procedure may therefore be more expensive to perform than one by the conventional procedure when a number of consolidation tests are to be performed. Also, when a number of consolidation tests are required for an investigation, the total number of calendar days required for the tests may not be less using the rapid procedure unless a number of experienced men are available to perform the tests simultaneously.

In this discussion the writer does not wish in any way to detract from the value of the author's ingenious rapid procedure for consolidation tests. The author's paper establishes a firm theoretical basis for a consolidation test procedure that many laboratories will find useful, not only to obtain test results quickly, but also to adjust standard test results for unequal time intervals for various load increments.

COMPUTATION OF THE STABILITY OF SLOPES[a]

Discussion by G. S. Sarkaria and N. L. Worth

G. S. SARKARIA,[1] J. M. ASCE and N. L. WORTH,[2] J. M. ASCE.—Engineers engaged in the design and analysis of earth embankments will find this paper a worthy attempt to relieve some of the monotony of repetitive time-consuming calculations. In the writers' experience, determination of the critical circle and minimum factor of safety for a particular slope and loading condition, in a homogeneous embankment, requires an average of ten trial circles. Using a planimeter, an experienced designer can check one circle in about two hours. Thus, nearly two to three man-days of continuous work should be allowed, as a minimum, for obtaining the critical safety factor for one set of conditions. Any method that can reduce this time requirement is a laudable contribution to design techniques. Mr. Meyer's analytical method is one such procedure.

The actual time-saving application of this paper is, however, limited to analysis of homogeneous embankments. In zoned embankments, where values of parameters c, w, and ϕ may vary from one zone to another, trials cannot be altogether avoided. The author has also pointed out this shortcoming of his method, and has explained ways and means of arriving at an answer with a minimum number of trials. Here it would be only fair to point out that a designer with a "feel" for the problem in hand and well-versed in the graphical method using a planimeter, is equally capable of finding short-cuts and arriving at an acceptable answer in a very short time.

Another process, which can find solutions to almost all variations of the problem of slope stability without need for the designer to resort to laborious and lengthy computations, is the use of electronic computers. Given the proper instructions, the computer can go through a designated number of trials, choosing the ones that have safety factors within a prescribed range, in a very short time. Programs for stability analysis of embankment by the "slip-circle" method have been developed by different organizations for different types of electronic computers. Such a program for the Bendix G-15D computer has been prepared by Mr. Michael A. C. Mann, J. M. ASCE, under the direction of Mr. E. Cristofano of the International Engineering Company, Inc. The versatility of this program can be judged by the fact that it is applicable to the following conditions:

1. Homogeneous embankments
2. Zoned embankments

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a. Proc. Paper 1824, October, 1958, by Otto H. Meyer.

1. Sr. Civ. Engr., International Eng. Co., Inc., San Francisco, Calif.
2. Soils Engr., Abbot A. Hanks, Inc., San Francisco. Formerly Soils Engr., International Engineering Co., Inc., San Francisco, Calif.

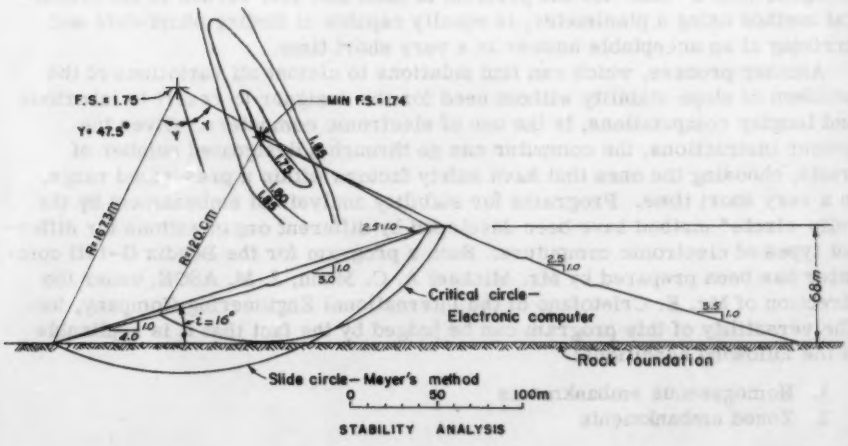
3. Uniform slopes
4. Variable or benched slopes
5. Embankments on rock foundations
6. Embankments on soil foundations
7. Sudden draw-down condition at upstream slope
8. Steady-state seepage condition at downstream slope
9. Combined circular and horizontal failure plane along rock foundation.

Different types of machines have different capacities to handle the number of variables or parameters. In most of them, the results are delivered to the engineer typed in a prearranged form. In the G-15, the factors of safety can be typed out for a graphic representation at the location of the center of the slide-circle, if desired. It can also check one circle at a time, typing out such significant data as the sliding and resisting forces, radius of circle, coordinates of the center, and width of unit strip assumed in the analysis. With this particular computer and program, the actual computation time per slide-circle is less than one minute for a homogeneous section and about two minutes for a zoned section.

Comparison between results obtained by the electronic computer and Mr. Meyer's method are quite interesting. In Fig. 1, upstream slope of Tres Marias Dam, Brazil, is shown. It is a homogeneous embankment of silty clay. The slope varies from 4.0 : 1.0 to 2.5 : 1.0 with a 3 m wide berm at about midheight. For this case the following parameters were assumed:

w, unit weight of clay, moist = 1.86 T/cu m
c, cohesion = 10.0 T/cu m
ϕ, angle of internal friction = 14^0

Height of dam at the section under analysis is 68 m and the dam is founded on rock.



STABILITY ANALYSIS

TRES MARIAS DAM

The critical circle obtained by the electronic computer which has a radius of 120.0 m is shown in Fig. 1. Iso-safety-factor contours are also shown. The minimum factor of safety, so obtained, is 1.74. The critical circle does not pass through either the toe or top of the slope. However, it does lie tangent to the rock foundation. For this particular case twenty-five circles were checked, primarily to study the variation in factor of safety with change in radius and location of the slide arc. It should be noted that although there is only one circle which gives the absolute minimum safety factor, a large number of circles with centers 50 m apart and radii from 95 m to 150 m show a factor of safety within 0.6 per cent of the "theoretical" minimum. These circles, like the critical circle, are tangential to the foundation rock.

When this dam is checked by the author's method, assuming the slope to be uniform and about 3.49 : 1.0 ($i = 16^{0}$), the minimum factor of safety is 1.75. This is within 0.6 per cent of the one determined by the electronic computer. The critical circle obtained by Mr. Meyer's method is also shown in Fig. 1. It represents what can be called a deep-seated slide, which is paradoxical since the plane of failure cannot extend through the bedrock foundation. Then what is the explanation for the close agreement in results? It can be coincidental.

Another noteworthy feature is the shape of the safety-factor contours. It appears that for flat slopes of homogeneous embankments, the locus of minimum factor (not the absolute minimum, of course) for all practical purposes is a long line. In other words, as pointed out before, any realistic circle with its center near this locus would give a factor of safety very close to the minimum. The circle corresponding to Mr. Meyer's solution, since it cuts into the rock foundation, should be considered a hypothetical case, obviously not applicable to this dam. This factor indicates the limitation of an analytical method, such as Mr. Meyer's method, but it does show that a designer can obtain an answer correct enough for his purpose with only a few trial circles. As in graphical analysis, after he has obtained the proper insight into characteristics of a particular embankment, the designer can arrange for the electronic computer to be more selective and obtain the minimum factor in a much shorter time.

It should also be kept in mind, in analysis of upstream slopes of earth dams, the minimum factor of safety that governs the selection of a particular slope generally occurs for the condition of sudden drawdown. In the graphical method, as well as when using the electronic computer, the designer's attention is concentrated on this condition. For example, in case of dam slope described before, the minimum factor of safety for steady state condition was 1.6 as compared to 1.2 for condition of rapid draw-down of 16 m from full reservoir. Naturally, therefore, a process that simplifies and reduces the work for condition of sudden draw-down is of great significance. The electronic computer is helpful in this situation also. The before-mentioned program for Bendix G-15 is capable of analysing for the draw-down condition based on the following assumption:

> If rapid drawdown is assumed to occur from Elevation X down to Elevation Y, then in the calculations for the factor of safety, the resisting forces are based on buoyant unit weights below Elevation X and on saturated unit weights above. The driving forces are based on saturated unit weights above Elevation Y and on buoyant unit weights below. The soil strength parameters used for this condition are those determined from "S"-tests performed on saturated specimens.

To consider the effect of pore pressure or uplift along an assumed slide plane in a cohesive material, it is quite convenient to superimpose the uplift force obtained graphically on the results of a critical circle obtained by the electronic computer.

The writers would like to point out another aspect of stability analysis of zoned embankments. Often the designer is not so much interested in finding the minimum factor of safety as in studying the change in the factor due to change in thickness and slopes of various zones. This is especially true when the section of the dam is in an evolutionary stage. The capability of the electronic computer in investigating a very large variety of embankment sections in a very short period is impressive indeed. With the computer doing all the monotonous computations for him, the designer has much more time at his disposal to evaluate the results and study their influence on stability and economy of the structure.

GLOSSARY OF TERMS AND DEFINITIONS IN SOIL MECHANICS[a]

Discussions by John A. Focht, Jr., Bernt Jakobson, R. G. Ahlvin
Paul A. Parisi

JOHN A. FOCHT, Jr.,[1] A. M. ASCE.—The committee has done an admirable job in preparing and organizing this glossary. Suggested modifications or revisions of some of the items are given below by their reference number.

111 Deviator Stress.—The definition for this item should be shifted to the group under No. 357, "Stress" leaving only the note "See Stress."

120, 121, 122 Active, At-Rest & Passive Earth Pressures.—From the standpoint of only these three terms, uniformity of subscripts is desirable and the use of a capital subscript to a lower case symbol seems odd. Therefore, even though there will be some duplication of symbols for other terms, the use of lower case subscripts for all three earth pressures would be desirable. Similar comments apply to Items 58, 59, 60 for coefficients of earth pressure.

197 Landslide.—A suggested slight revision of the definition for this item is "The failure of a sloped bank of soil in which the movement of the soil mass takes place primiarily along a single surface of sliding."

234 Optimum Moisture Content.—The basic symbol used for water content is a lower case w. Therefore, the second symbol should be w_o. This comment also applies in general to Item 226.

311 Seepage Force.—Under some circumstances, the use of a seepage force per unit volume is advantageous. Therefore, two symbols, J and j, for total and unit forces should be established.

324 Silt and

325 Silt Size.—The comments for these two items should be very parallel to those for Items 49 and 50, Clay and Clay Size.

> Silt. Fine-grained soil or the fine-grained portion of soil that is non-plastic to very slightly plastic and that exhibits little or no strength when air dried. The term has been used to designate the percentage of soil finer than 0.02 mm and coarser than 0.002 mm (0.05 and 0.005 in some cases), but it is strongly recommended that this usage be discontinued, since there is ample evidence that from an engineering standpoint the properties described in the above definition are more important.
>
> Silt Size. That portion of soil finer than 0.02 mm and coarser than 0.002 mm (0.05 mm and 0.005 mm in some cases.) (See discussion under "Silt")

a. Proc. Paper 1826, October, 1958, by the Committee on Glossary of Terms and Definitions.

1. Chf. Design Engr., McClelland Engineers, Inc., Houston, Tex.

327 Skin Friction.—The quotation marks around "structure" should be omitted because "structure" in this instance refers to a portion of a foundation rather than to "soil structure."

384 Toughness Index.—No definition is given.

One additional term and its corresponding symbol which is frequently used is "Overburden Pressure, p_o." This term is parallel to Item 275, "Preconsolidation Pressure" and can be defined as "The existing 'effective pressure' acting on a 'soil'."

In regard to the terms considered by the committee to be controversial, it is the writer's opinion that the relatively simple definitions presented are entirely adequate and are sufficiently complete for the purpose of the glossary. The more complex definitions used in some papers, for example for Items 15 & 67—"angle of internal friction" and "cohesion", would not be compatible with the short concise definitions of the other terms. Whenever special definitions are required, they can be established; but for general usage, the definitions as given probably conform to the understanding of the various terms as held by most engineers.

BERNT JAKOBSON.[1]—This report has proven to be most interesting. The committee's effort to reach a better agreement on this subject is highly appreciated.

As the committee is no doubt aware, there was appointed a special international committee on notations and symbols in London in 1957. The writer, being a member of that committee, is hopeful that this report will not be made final until it has been discussed on an international level and also treated by the international committee.

For the information of the committee and of interested readers, there follows herewith the "Proposed List of Symbols" prepared by the Swedish Geotechnical Society to the London Conference. This list is not a final one but is offered for comment and review by the committee.

1. Chmn., Swedish Geotechnical Society, Stockholm, Sweden.

Proposed List of Symbols

A. Pure Geotechnical Symbols

Symbol	Can be further explained by index	Quantity	Dimension[1]	Note
β		Angle between earth and horizon	-	
φ		Angle of shearing resistance	-	$\tau_f = c + \sigma \tan\varphi$
$\bar{\varphi}$		Apparent angle of shearing resistance	-	$\tau_f = \bar{c} + \bar{\sigma} \tan\bar{\varphi}$
φ_o		Angle of internal friction ("true angle")	-	
δ		Angle of wall friction	-	
N	N_c, $N\gamma$, N_q	Bearing capacity factor	-	
γ		Bulk density	t/m^3	
$\varkappa$		Coefficient of cohesion increase	-	"Hvorslev's constant"
c_v		Coefficient of consolidation	cm^2/s	
K	K_a, K_p, K_o	Coefficient of earth pressure	-	
k		Coefficient of permeability	cm/s or $cm^4/kg\ s$	
k_s		Coefficient of subgrade reaction	kg/cm^3	
m_v		Coefficient of volume compressibility	cm^2/kg	

K	K_a, K_p, K_o	Coefficient of earth pressure		
k		Coefficient of permeability	cm/s or $cm^4/kg\ s$	
k_s		Coefficient of subgrade reaction	kg/cm^3	
m_v		Coefficient of volume compressibility	cm^2/kg	
c		Cohesion (in terms of total stresses)	kg/cm^2	$\tau_f = c + \sigma \tan \varphi$
$\bar{c}$		Cohesion (in terms of effective stresses)	kg/cm^2	$\tau_f = \bar{c} + \bar{\sigma} \tan \bar{\varphi}$
λ		Compressibility ratio (Skempton's constant)		$\lambda = \frac{E_c}{E_s} \frac{1 - 2/\mu_s}{1 - 2/\mu_c}$
C	C_c, C_s	Compression index	-	
I_C		Consistency index (relative consistency)		$I_C = \frac{w_L - w}{w_L - w_P}$
U		Degree of consolidation	%	
S_w		Degree of saturation	%	$S_w = \frac{w}{100\ e} \frac{\gamma_s}{\gamma_w}$
γ_s		Density of solid particles	t/m^3	
γ_w		Density of water	t/m^3	
γ_d		Dry density	t/m^3	
F	F_c, F_φ	Factor of safety	-	
h_c		Height of capillary rise	m	

1) t, kg, g can be replaced by any dimension for weight
cm, m " " " " " " " length,
and so on.

Symbol	Can be further explained by index	Quantity	Dimension	Note
w_L		Liquid limit	%	Per cent of dry weight
I_L		Liquidity index	-	$I_L = 1 - I_C$
M	M_c, M_s	Modulus of compressibility	kg/cm^2	$M = \frac{1}{m_v}$
w_P		Plastic limit	%	
I_P		Plasticity index	%	$I_P = w_L - w_P$
u	u_e, u_z, u_w [2]	Pore water pressure	kg/cm^2	
n		Porosity	%	
p_c		Preconsolidation pressure	kg/cm^2	
D_r		Relative density	-	$D_r = \frac{e_{max} - e}{e_{max} - e_{min}}$
S_t		Sensitivity	-	
Δ [3]	ΔD, ΔH etc.	Settlement	m	
w_S		Shrinkage limit	%	Per cent of dry weight
A		Skempton's pore pressure coefficient		
B		Skempton's pore pressure coefficient		
G_s		Specific gravity of soil particles	g/cm^3	
γ		Submerged density	t/m^3	

B		Skempton's pore pressure coefficient		
G_s		Specific gravity of soil particles	g/cm^3	
γ		Submerged density	t/m^3	
T_v		Time factor	-	$T_v = \frac{c_v t}{H^2}$
P	P_a, P_p, P_o	Total earth pressure	t, t/m	
p	p_a, p_p, p_o	Unit earth pressure	t/m^2	
e		Void ratio	-	
w		Water content	%	Per cent of dry weight

Note: A horizontal line above a symbol, e.g. $\bar{\sigma}$, means effective.

2) When necessary we distinguish between excess pressure u_e, hydrostatic pressure u_z, and total pressure u_w; thus

$$u_w = u_e + u_z$$

3) The symbol Δ should always be followed by another symbol, thus ΔD, ΔH etc.

B. Some other Symbols often used in Soil Mechanics

Symbol	Can be further explained by index	Quantity	Dimension	Note
A		Area	m^2, m^2/m	
i		Hydraulic gradient	cm/cm, kg/cm^3	
E	E_c, E_s	Modulus of elasticity	kg/cm^2	
G		Modulus of rigidity	kg/cm^2	
M		Moment	tm, tm/m	
I		Moment of inertia	m^4, m^4/m	
ε		Normal strain	cm/cm	
σ		Normal stress	kg/cm^2	
μ		Poisson's ratio	-	
W		Section modulus	m^3, m^3/m	
γ		Shear strain	-	
τ_f		Shear strength	kg/cm^2	
τ		Shear stress	kg/cm^2	
t		Time	s	
v		Velocity	cm/s	
V		Volume	m^3, m^3/m	

R. G. AHLVIN,[1] M. ASCE.—The preface to the basic report indicates the rather extensive effort which has been put into the preparation of these definitions and mentions the problem of gaining coordinated agreement between the two committees involved and their individual members. The purpose of this discussion is to point out an ASTM subcommittee action required in connection with final arrangements for ASTM publication of the definitions as a "Tentative" which, because of timing, could not be properly coordinated.

The required action concerned the definition for "Earth Pressure" and relates to the use of the term (with symbols) for both pressure and force. The problem was aggravated in the ASTM galley proofs by an editorial change which substituted "Unit Earth Pressure" and "Total Earth Pressure" for the single words "Unit" and "Total" beside the lower- and upper-case symbols above the definition. This added the confusion of a comparison with soil mechanics use of total pressure as the sum of hydrostatic and intergranular pressure.

It was decided that since most soil mechanics textbooks treat the term "Earth Pressure" in the general or common dictionary sense of "a pressing" by earth and treat it variously as force or pressure (unit force), it must be so treated in a definition for soil mechanics use. Also, to avoid undesirable connotations, it was decided to use the terms "Pressure Symbol" and "Force Symbol" to designate, respectively, the lower- and upper-case symbols for earth pressure.

This action had to be taken initially with almost no coordination. It has since been endorsed by most of the ASTM and some of the ASCE Committee members. The problem is being reviewed here to give it wide attention and to provoke comment.

PAUL A. PARISI,[2] A. M. ASCE.—Prior to putting the Committee's report to use, it was found necessary to arrange the suggested letter symbols in alphabetical order. Such an arrangement revealed a multiplicity of definitions for some letter symbols. It is sincerely hoped that the committee can resolve some of the conflict in this list:

A_r = area ratio of sampling spoon, sampler, or sampling unit (24);

a = area of influence of a well (23);

a_v = coefficient of compressibility (coefficient of compression) (55);

B = liquidity index (water-plasticity ratio) (relative water content) (206)*;

C_a = total adhesion (6);

C_c = compression index (77);

C_r = relative consistency (295)*;

C_u = coefficient of uniformity (64);

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2. Editor, Technical Publications, ASCE, New York, N. Y.

CBR = California Bearing Ratio (40);

CME = centrifuge moisture equivalent (226)*;

c = cohesion (67);

c_a = unit adhesion (6);

c_v = coefficient of consolidation (56);

D = equivalent diameter (132);

D_d = relative density (296);

D_e = effective diameter (123)*;

D_{10} = effective diameter (123)*;

E = modulus of elasticity (218)*;

e = void ratio (409);

e_c = critical void ratio)410);

F = freezing index (157);

$\bar{F}$ = effective force (125);

F_w = flow index (147)*;

FME = field moisture equivalent (227);

f = stress (357)*;

f = total stress (366)*;

f = skin friction (327);

$\bar{f}$ = effective stress (effective pressure) (intergranular pressure) (358)*;

f' = wall friction (412);

G_a = air-space ratio (9);

G_a = apparent specific gravity (346)*;

G_m = bulk specific gravity (specific mass gravity) (347)*;

G_s = specific gravity of solids (345)*;

G_v = air-void ratio (10);

H_c = critical height (98);

h = capillary head (44);

h_c = capillary rise (height of capillary rise) (45);

Δh = potential drop (274);

I = influence value (186);

I_c = relative consistency (295)*;

I_f = flow index (147)*;

I_L = liquidity index (water-plasticity index) (relative water content) (206)*;

I_p = plasticity index (271)*;

I_R = remolding index (299);

I_T = toughness index (384)*;

I_w = plasticity index (271)*;

i = hydraulic gradient (180)*;

i_c = critical hydraulic gradient (181);

J = seepage force (311);

K = coefficient of earth pressure (57);

K_A = coefficient of earth pressure, active (58);

K_O = coefficient of earth pressure, at rest (59);

K_P = coefficient of earth pressure, passive (60);

k = coefficient of permeability (62);

k = coefficient of subgrade reaction (modulus of subgrade reaction) (63);

L_E = linear expansion (202);

L_S = linear shrinkage (203);

L_w = liquid limit (204)*;

LL = liquid limit (204)*;

M = modulus of elasticity (modulus of deformation) (218)*;

m_v = coefficient of volume compressibility (modulus of volume change) (66);

N = penetration resistance (standard penetration resistance) (Proctor penetration resistance) (245)*;

N_s = stability factor (stability number) (349);

N_ϕ = flow value (151);

n = porosity (273);

n = concentration factor (79);

n_e = effective porosity (126);

OMC = optimum moisture content (optimum water content) (234)*;

P = earth pressure, total (119);

P_A = earth pressure, active, total (120);

P_a = allowable pile-bearing load (12)*;

P_O = earth pressure, at rest, total (121);

P_P = earth pressure, passive, total (122);

P_p = bearing capacity (of a pile) (29)*;

P_w = plastic limit (268)*;

PI = plasticity index (271)*;

PL = plastic limit (268)*;

p = contact pressure (92);

p = earth pressure, unit (119);

p = pressure (276);

p = stress (357);

p = normal stress (360);

p_A = earth pressure, active, unit (20);

p_a = allowable bearing value (allowable soil pressure) (11)*;

p_c = compressive strength (unconfined compressive strength) (78)*;

p_c = preconsolidation pressure (prestress) (275);

p_O = earth pressure, at rest, unit (121);

p_P = earth pressure, passive, unit (122);

p_R = penetration resistance (standard penetration resistance) (Proctor penetration resistance) (245)*;

pH = an index of the acidity or alkalinity of a soil (254);

Q_a = allowable pile-bearing load (12)*;

Q_P = bearing capacity (of a pile) (29)*;

q_a = allowable bearing value (allowable soil pressure) (11)*;

q_o = ultimate bearing capacity (389)*;

q_u = compressive strength (unconfined compressive strength) (78)*;

q_{ult} = ultimate bearing capacity (389)*;

R = shrinkage ratio (323);

R_w = liquidity index (water-plasticity ratio) (relative water content) (206)*;

S = per cent saturation (degree of saturation (degree of saturation) (249);

S_a = apparent specific gravity (346)*;

S_m = bulk specific gravity (specific mass gravity) (347)*;

S_s = specific gravity of solids (345)*;

S_t = remolding sensitivity (sensitivity ratio) (300);

SI = shrinkage index (321);

SL = shrinkage limit (322);

s = hydraulic gradient (180)*;

s = shear strength (319);

s = shear stress (shearing stress) (tangential stress) (365)*;

T = time factor (380)*;

T_v = time factor (380)*;

T_w = sticky limit (353);

T_w = toughness index (384)*;

U = degree of consolidation (per cent consolidation) (106);

U = total uplift (403);

U_z = consolidation ratio (89);

u = unit uplift (403);

u = excess hydrostatic pressure (hydrostatic excess pressure) (183)*;

u = neutral stress (pore pressure) (pore water pressure) (359)*;

$\bar{u}$ = excess hydrostatic pressure (hydrostatic excess pressure) (183)*;

u_o = hydrostatic pressure (182);

u_w = neutral stress (pore pressure) (pore water pressure) (359)*;

V_s = volumetric shrinkage (411);

v = discharge velocity (114);

v_1 = seepage velocity (313)*;

v_s = seepage velocity (313)*;

W_c = centrifuge moisture equivalent (226)*;

W_o = optimum moisture content (optimum water content) (234)*;

w = moisture content (water content) (223);

w_c = hygroscopic capacity (hygroscopic coefficient) (184);

w_H = hygroscopic water content (185);

w_L = liquid limit (204)*;

w_p = plastic limit (268)*;

α = angle of obliquity (16)*;

α = angle of repose (17);

β = angle of obliquity (6)*;

γ = unit weight (394);

γ_d = dry unit weight (395)*;

γ_e = effective unit weight (396);

γ_G = saturated unit weight (398)*;

γ_m = submerged unit weight (buoyant unit weight) (399)*;

γ_m = wet unit weight (mass unit weight) (401)*;

γ_{max} = maximum unit weight (397);

γ_o = dry unit weight (395)*;

γ_{sat} = saturated unit weight (398)*;

γ_{sub} = submerged unit weight (buoyant unit weight) (399)*;

γ_w = unit weight of water (400);

γ_{wet} = wet unit weight (mass unit weight) (401)*;

γ_z = zero air voids unit weight (402);

γ' = submerged unit weight (buoyant unit weight)(399)*;

Δ = deviator stress (111)*;

δ = angle of external friction (angle of wall friction) (14);

ϵ = strain (355);

θ = angle of obliquity (16)*;

μ = coefficient of viscosity (coefficient of absolute viscosity) (65);

σ = normal stress (360)*;

σ = deviator stress (111)*;

σ = stress (357)*;

σ = total stress (366)*;

σ_1 = major principal stress (362);

σ_2 = intermediate principal stress (364);

σ_3 = minor principal stress (363);

$\bar{\sigma}$ = effective stress (effective pressure (intergranular pressure) (358);

τ = shear stress (shearing stress) (tangential stress) (365);

ϕ = angle of internal friction (15);

ψ = angle of obliquity (6)*;

In this list the number in parentheses at the end of the definition corresponds to the item number in the Committee's report. An asterisk is used to identify a letter symbol that is one of several used for the same term.

It would, of course, be too much to expect all areas to be represented by A's, all coefficients by C's, diameters by D's, forces by F's, indexes by I's, pressures by P's, ratios by R's, unit weights by γ's, and stresses by σ's. But it would be helpful if the committee could, in cases of multiple definitions for the same letter symbol, attempt to reconcile the differences and eliminate the conflict.

For example, C should not represent adhesion, compression index, relative consistency, and coefficient of uniformity. Similar conflicts occur for c, D, F, f, G, I, k, L, N, n, P, p, R, S, s, T, U, u, w, and α. It is true, of course, that subscripts and superscripts are used for some of these symbols in an

attempt to resolve the conflicts, but where several symbols are used for the same physical quantity, the symbol used should be that which does not conflict with another usage of a letter symbol.

PROCEEDINGS PAPERS

The technical papers published in the past year are identified by number below. Technical-division sponsorship is indicated by an abbreviation at the end of each Paper Number, the symbols referring to: Air Transport (AT), City Planning (CP), Construction (CO), Engineering Mechanics (EM), Highway (HW), Hydraulics (HY), Irrigation and Drainage (IR), Pipeline (PL), Power (PO), Sanitary Engineering (SA), Soil Mechanics and Foundations (SM), Structural (ST), Surveying and Mapping (SU), and Waterways and Harbors (WW), divisions. Papers sponsored by the Department of Conditions of Practice are identified by the symbols (PP). For titles and order coupons, refer to the appropriate issue of "Civil Engineering." Beginning with Volume 82 (January 1956) papers were published in Journals of the various Technical Divisions. To locate papers in the Journals, the symbols after the paper number are followed by a numeral designating the issue of a particular Journal in which the paper appeared. For example, Paper 1859 is identified as 1859 (HY 7) which indicates that the paper is contained in the seventh issue of the Journal of the Hydraulics Division during 1958.

VOLUME 84 (1958)

APRIL: 1580(EM2), 1581(EM2), 1582(HY2), 1583(HY2), 1584(HY2), 1585(HY2), 1586(HY2), 1587(HY2), 1588 (HY2), 1589(IR2), 1590(IR2), 1591(IR2), 1592(SA2), 1593(SU1), 1594(SU1), 1595(SU1), 1596(EM2), 1597(PO2), 1598(PO2), 1599(PO2), 1600(PO2), 1601(PO2), 1602(PO2), 1603(HY2), 1604(EM2), 1605(SU1)[c], 1606(SA2), 1607(SA2), 1608(SA2), 1609(SA2), 1610(SA2), 1611(SA2), 1612(SA2), 1613(SA2), 1614(SA2)[c], 1615(IR2)[c], 1616 (HY2)[c], 1617(SU1), 1618(PO2)[c], 1619(EM2)[c], 1620(CP1).

MAY: 1621(HW2), 1622(HW2), 1623(HW2), 1624(HW2), 1625(HW2), 1626(HW2), 1627(HW2), 1628(HW2), 1629 (ST3), 1630(ST3), 1631(ST3), 1632(ST3), 1633(ST3), 1634(ST3), 1635(ST3), 1636(ST3), 1637(ST3), 1638(ST3), 1639(WW3), 1640(WW3), 1641(WW3), 1642(WW3), 1643(WW3), 1644(WW3), 1645(SM2), 1646(SM2), 1647 (SM2), 1648(SM2), 1649(SM2), 1650(SM2), 1651(HW2), 1652(HW2)[c], 1653(WW3)[c], 1654(SM2), 1655(SM2), 1656(ST3)[c], 1657(SM2)[c].

JUNE: 1658(AT1), 1659(AT1), 1660(HY3), 1661(HY3), 1662(HY3), 1663(HY3), 1664(HY3), 1665(SA3), 1666 (PL2), 1667(PL2), 1668(PL2), 1669(AT1), 1670(PO3), 1671(PO3), 1672(PO3), 1673(PL2), 1674(PL2), 1675 (PO3), 1676(PO3), 1677(SA3), 1678(SA3), 1679(SA3), 1680(SA3), 1681(SA3), 1682(SA3), 1683(PO3), 1684 (HY3), 1685(SA3), 1686(SA3), 1687(PO3), 1688(SA3)[c], 1689(PO3)[c], 1690(HY3)[c], 1691(PL2)[c].

JULY: 1692(EM3), 1693(EM3), 1694(ST4), 1695(ST4), 1696(ST4), 1697(SU2), 1698(SU2), 1699(SU2), 1700(SU2), 1701(SA4), 1702(SA4), 1703(SA4), 1704(SA4), 1705(SA4), 1706(EM3), 1707(ST4), 1708(ST4), 1709(ST4), 1710 (ST4), 1711(ST4), 1712(ST4), 1713(SU2), 1714(SA4), 1715(SA4), 1716(SU2), 1717(SA4), 1718(EM3), 1719 (EM3), 1720(SU2), 1721(ST4)[c], 1722(ST4), 1723(ST4), 1724(EM3)[c].

AUGUST: 1725(HY4), 1726(HY4), 1727(SM3), 1728(SM3), 1729(SM3), 1730(SM3), 1731(SM3), 1732(SM3), 1733 (PO4), 1734(PO4), 1735(PO4), 1736(PO4), 1737(PO4), 1738(PO4), 1739(PO4), 1740(PO4), 1741(PO4), 1742 (PO4), 1743(PO4), 1744(PO4), 1745(PO4), 1746(PO4), 1747(PO4), 1748(PO4), 1749(PO4).

SEPTEMBER: 1750(IR3), 1751(IR3), 1752(IR3), 1753(IR3), 1754(IR3), 1755(ST5), 1756(ST5), 1757(ST5), 1758(ST5), 1759(ST5), 1760(ST5), 1761(ST5), 1762(ST5), 1763(ST5), 1764(ST5), 1765(WW4), 1766(WW4), 1767(WW4), 1768(WW4), 1769(WW4), 1770(WW4), 1771(WW4), 1772(WW4), 1773(WW4), 1774(IR3), 1775 (IR3), 1776(SA5), 1777(SA5), 1778(SA5), 1779(SA5), 1780(SA5), 1781(WW4), 1782(SA5), 1783(SA5), 1784 (IR3)[c], 1785(WW4)[c], 1786(SA5)[c], 1787(ST5)[c], 1788(IR3), 1789(WW4).

OCTOBER: 1790(EM4), 1791(EM4), 1792(EM4), 1793(EM4), 1794(EM4), 1795(HW3), 1796(HW3), 1797(HW3), 1798(HW3), 1799(HW3), 1800(HW3), 1801(HW3), 1802(HW3), 1803(HW3), 1804(HW3), 1805(HW3), 1806 (HY5), 1807(HY5), 1808(HY5), 1809(HY5), 1810(HY5), 1811(HY5), 1812(SM4), 1813(SM4), 1814(ST6), 1815 (ST6), 1816(ST6), 1817(ST6), 1818(ST6), 1819(ST6), 1820(ST6), 1821(ST6), 1822(EM4), 1823(PO5), 1824 (SM4), 1825(SM4), 1826(SM4), 1827(ST6)[c], 1828(SM4)[c], 1829(HW3)[c], 1830(PO5)[c], 1831(EM4)[c], 1832(HY5)[c]

NOVEMBER: 1833(HY6), 1834(HY6), 1835(SA6), 1836(ST7), 1837(ST7), 1838(ST7), 1839(ST7), 1840(ST7), 1841(ST7), 1842(SU3), 1843(SU3), 1844(SU3), 1845(SU3), 1846(SU3), 1847(SA6), 1848(SA6), 1849(SA6), 1850(SA6), 1851(SA6), 1852(SA6), 1853(SA6), 1854(ST7), 1855(SA6)[c], 1856(HY6)[c], 1857(ST7)[c], 1858 (SU3)[c].

DECEMBER: 1859(HY7), 1860(IR4), 1861(IR4), 1862(IR4), 1863(SM5), 1864(SM5), 1865(ST9), 1866(ST9), 1867 (ST8), 1868(PP1), 1869(PP1), 1870(PP1), 1871(PP1), 1872(PP1), 1873(WW5), 1874(WW5), 1875(WW5), 1876 (WW5), 1877(CP2), 1878(ST8), 1879(ST8), 1880(HY7)[c], 1881(SM5)[c], 1882(ST8)[c], 1883(PP1)[c], 1884(WW5)[c], 1885(CP2)[c], 1886(PO6), 1887(PO6), 1888(PO6), 1889(PO6), 1890(HY7), 1891(PP1).

VOLUME 85 (1959)

JANUARY: 1892(AT1), 1893(AT1), 1894(EM1), 1895(EM1), 1896(EM1), 1897(EM1), 1898(EM1), 1899(HW1), 1900(HW1), 1901(HY1), 1902(HY1), 1903(HY1), 1904(HY1), 1905(PL1), 1906(PL1), 1907(PL1), 1908(PL1), 1909(ST1), 1910(ST1), 1911(ST1), 1912(ST1), 1913(ST1), 1914(ST1), 1915(ST1), 1916(AT1)[c], 1917(EM1)[c], 1918(HW1)[c], 1919(HY1)[c], 1920(PL1)[c], 1921(SA1)[c], 1922(ST1)[c], 1923(EM1), 1924(HW1), 1925(HW1), 1926 (PL1), 1927(HW1), 1928(HW1), 1929(SA1), 1930(SA1), 1931(SA1), 1932(SA1).

FEBRUARY: 1933(HY2), 1934(HY2), 1935(HY2), 1936(SM1), 1937(SM1), 1938(ST2), 1939(ST2), 1940(ST2), 1941(ST2), 1942(ST2), 1943(ST2), 1944(ST2), 1945(HY2), 1946(PO1), 1947(PO1), 1948(PO1), 1949(PO1), 1950(HY2)[c], 1951(SM1)[c], 1952(ST2)[c], 1953(PO1)[c], 1954(CO1), 1955(CO1), 1956(CO1), 1957(CO1), 1958 (CO1), 1959(CO1).

MARCH: 1960(HY3), 1961(HY3), 1962(HY3), 1963(IR1), 1964(IR1), 1965(IR1), 1966(IR1), 1967(SA2), 1968 (SA2), 1969(ST3), 1970(ST3), 1971(ST3), 1972(ST3), 1973(ST3), 1974(ST3), 1975(ST3), 1976(WW1), 1977 (WW1), 1978(WW1), 1979(WW1), 1980(WW1), 1981(WW1), 1982(WW1), 1983(WW1), 1984(SA2), 1985(SA2)[c], 1986(IR1)[c], 1987(WW1)[c], 1988(ST3)[c], 1989(HY3)[c].

APRIL: 1990(EM2), 1991(EM2), 1992(EM2), 1993(HW2), 1994(HY4), 1995(HY4), 1996(HY4), 1997(HY4), 1998 (SM2), 1999(SM2), 2000(SM2), 2001(SM2), 2002(ST4), 2003(ST4), 2004(ST4), 2005(ST4), 2006(PO2), 2007 (HW2)[c], 2008(EM2)[c], 2009(ST4)[c], 2010(SM2)[c], 2011(SM2)[c], 2012(HY4)[c], 2013(PO2)[c].

c. Discussion of several papers, grouped by divisions.

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SM 2

APRIL 1959 — 18 NO. SM 2
VOLUME 85 PART 2

Your attention is invited

NEWS OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION OF ASCE

JOURNAL OF THE SOIL MECHANICS AND FOUNDATIONS DIVISION
PROCEEDINGS OF THE AMERICAN SOCIETY OF CIVIL ENGINEERS

DIVISION ACTIVITIES

SOIL MECHANICS AND FOUNDATIONS DIVISION

Proceedings of the American Society of Civil Engineers

NEWS

April, 1959

SAN FRANCISCO NEWS

On January 21 the Soil Mechanics and Foundations Engineering Division was addressed by Professor H. B. Seed of the University of California at Berkeley on "Strength Characteristics of Compacted Soils". Following Professor Seed's talk the group toured the University's Soil Mechanics and Bituminous Materials Laboratory. Professor Seed's presentation included a discussion of the structure of compacted clays, the effect of structure on the engineering properties of laboratory-prepared soil samples, and the relationship of different methods of laboratory compaction, soil structure and physical properties such as strength, shrinkage and expansion.

The tour of the laboratory included demonstrations of repeated loading tests in soils, pore water pressure measurements in triaxial compression tests, determination of soil structure by optical methods and various tests on bituminous paving mixtures.

Prior to the technical program a business meeting was held, at which time new officers were elected. For 1959 the officers are: Chairman, Raymond Lundgren; Vice-Chairman, R. D. Darragh; Secretary, C. K. Chan.

On February 17 the entire San Francisco Section heard a talk by Dr. Gerald W. Johnson, Head of "Operation Plowshare" at the University of California Radiation Laboratory, Livermore, California. "Operation Plowshare" is the atomic energy program to demonstrate peaceful uses of atomic explosives. Dr. Johnson explained the uses of nuclear explosives creating harbors and for other large scale earth moving operations.

On January 30 and 31 a program entitled "Engineering and Geological Aspects of Urban Hillside Site Development" was presented through the auspices of the University of California in Los Angeles. This conference included the following sessions.

1. Materials and Processes.
2. Case Histories
3. Problems and Workshops

The conference was designed to meet the needs of the following groups of people: Civil Engineers, Geologists, Building Inspectors, Subdividers,

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Grading Contractors, Structural Engineers, Architects, Researchers, City and County Planners, Insurance Adjusters, and Landscape Architects. Inquiries as to proceedings or other publications should be made to the University Extension, University of California, Los Angeles 24, California.

NEW $5000 RESEARCH FELLOWSHIP AVAILABLE

A new Fellowship Grant in the field of Civil Engineering research has just been announced by the American Society of Civil Engineers. Rules to guide applicants are being published in the 1959 ASCE Official Register, as follows:

The ASCE Research Fellowship was established in 1958 by the Board of Direction, for the purpose of aiding in the creation of new knowledge for the benefit and advancement of the science and profession of civil engineering. The grant is made from current income of the Society.

1. The grant, in the amount of $5000, is made annually.
2. Applicants must be members of the Society in any grade of membership, be a citizen of the United States, and have been graduated from an accredited curriculum.
3. Selection for the award of the grant is made by the Committee on Fellowships, Scholarships, Grants and Bequests on the basis of the following:
   a. Transcripts of scholastic records.
   b. Ability to conceive and explore original ideas (letters of recommendation will be considered only when they contribute specific information on this ability.)
   c. Description of proposed research and its objectives, including a statement from the institution at which the research is to be done that the applicant and proposed research are acceptable to the institution. Preference is given to research that is basic in nature and concept, rather than applied, developmental, or designed to extend or elaborate information. Research requiring an extensive testing program will not be considered.
4. Applications in specified forms will be received to March 15, 1959, and awards made by May 15, 1959, for a twelve-month period beginning not later than the following October 1.
5. The Research Fellow is required to devote full time to his proposed research during the tenure of the fellowship.
6. The Research Fellow is required to submit a report on his research, suitable for publication, to the Secretary of the Society on completion of the tenure of his fellowship.

MINNESOTA CONFERENCE

The Seventh Annual Conference on Soils and Mechanics and Foundation Engineering was held Thursday, March 26, 1959 at the University of Minnesota, in cooperation with the Northwestern Section of the American Society of Civil Engineers.

The program this year dealt with fundamentals of soil exploration and testing, correlations of soil test data to design and bearing capacity of structures

on compacted soils. Consideration was also given to principles of dewatering foundations and the effect of ground water level variations on structures.

The course was planned especially for engineers and architects in structural and highway fields, but was open to all persons with an interest in design and construction problems concerning soils.

Following are a list of talks that were presented:

Fundamentals of Soil Exploration and Testing . . . . . Donald E. Johnson
Correlations of Soil Test Data to Design
Relating to Structures . . . . . E. L. Gardner
Relating to Highways and Airfields . . . . . Richard P. Braun
Bearing Capacity of Structures on Compacted Soils . Ralph B. Peck
Principles of Dewatering Foundations and Effect
of Ground Water Level Variations of Structures. . . Adolph F. Meyer

Fees for the conference were: registration, $3.00; tuition, $4.50; total, $7.50. The cost of issuing proceedings for the conference was included in the fees.

M.I.T. PROGRAM ON SHEAR STRENGTH OF SOILS

The Massachusetts Institute of Technology announces a two-week summer program on "The Shear Strength of Soils" to be given from August 31 through September 11, 1959. The program is designed to provide practicing engineers, researchers, and educators with an up-to-date understanding of the fundamental nature and use of shear strength in engineering problems.

The program will emphasize the selection of the proper shear strength to use in soil engineering problems. It will present the current state of knowledge on such questions as: "When is it correct to use the results of unconfined compression tests: When are the more complicated triaxial tests required? Under what conditions can field tests be best utilized?" Since answers to such problems are predicated on an understanding of the role which effective stresses and pore water pressures play, considerable time will be devoted to a treatment of the effective stress principle. Not only does the effective stress principle help the engineer to select the proper shear strength for the simpler problems, but it aids him in handling the very difficult problems in which theoretical principles become inadequate and judgment must be used to arrive at a safe and economical solution.

A partial list of the topics to be covered is: (1) the effective stress principle and the role of effective stress in determining shear strength; (2) factors affecting changes in effective stresses within soil masses; (3) examples using unconfined compression and using drained triaxial or direct shear tests to solve stability problems; limitations of these methods will be noted; (4) problems in which special methods must be used to determine shear strength; problems involving partially saturated soils will be included; (5) methods of laboratory and field shear strength measurements; (6) nature of clay and the mechanism of shear strength in clays; (7) case studies of embankments on soft clay, earth dam construction, and problems involving stiff-fissured clays; (8) the use of shear strength for the design of pile foundations and pavements; (9) special topics, such as the effect of strain rate upon shear strength. Emphases will be given to fine-grained soils.

The lectures will be supplemented by problem sessions and discussion periods. Each participant will have the opportunity to use the shear testing apparatus in M.I.T.'s new and well-equipped Soil Engineering Laboratory.

The program will be under the direction of Dr. T. William Lambe, Associate Professor and Head of the Soil Engineering Division. Other members of the M.I.T. staff who will participate in the program include: Professors R. V. Whitman and J. E. Roberts and Messrs. C. C. Ladd, R. M. Harkness, and A. M. Richardson, Dr. A. W. Skempton, Professor and Head of Civil Engineering, Imperial College of Science and Technology, University of London, and Dr. Laurits Bjerrum, Director of the Norwegian Geotechnical Institute and Visiting Professor at M.I.T., will discuss the effective stress principle and field jobs involving the construction of earth dams on soft clay foundations, the stability of cuttings and retaining walls in stiff fissured clays, and the long-term stability of natural slopes in over-consolidated clays. Dr. Karl Terzaghi, Professor of the Practice of Civil Engineering, Emeritus, Harvard University, and Lecturer at M.I.T., will present his views on the use of shear strength in practice.

For further information regarding application and tuition, please write to the Director of summer Sessions, M.I.T., Cambridge 39, Massachusetts. (Note that the two-week period includes Labor Day).

COLORADO SOIL MECHANICS CONFERENCE

Soil mechanics, an important factor in determining construction costs in the Rocky Mountain area, will be discussed April 23 at a one-day Soil Mechanics Conference, to be held at the Colorado School of Mines. The program will feature five papers aimed at theoretical and practical treatments of expansive soils.

The conference is co-sponsored by the Soil Mechanics and Foundations Division of the Colorado Section of the American Society of Civil Engineers and the School of Mines. The program will begin at 8:45 A.M., in the Mines Library. Approximately 250 engineers and architects are expected to attend the conference, first of its kind ever to be held in this region.

The conference will deal mainly with practical applications of soil mechanics theories. In recent years shrinking and swelling soils have been recognized as a major contributor to critical design problems in engineering for home, highways, earth structures, and mines. Because of the small amount of knowledge now available in the field of soil mechanics, damage to completed structures has been rapidly increasing. Much of this damage has occurred west of the Mississippi River where expansive soils have ruined millions of dollars of construction.

The speakers for the program include Chester McDowell, Supervising Engineer for Soils, the Texas Highway Department; Raymond F. Dawson, Professor of Civil Engineering at the University of Texas; W. G. Holtz, Chief of the Earth Laboratory Branch of the U. S. Bureau of Reclamation; R. E. Means, Professor of Architecture and Civil Engineering at Oklahoma State University; T. William Lambe; Professor of Soil Mechanics at the Massachusetts Institute of Technology; and R. V. Whitman, Professor of Civil Engineering at M.I.T.

In their papers the speakers will deal with engineering and physical properties of expansive soils, practices employed in design for such soils and testing

procedures used to aid in the design of structures for this type of material.

MIDWESTERN CONFERENCE

Announcement was made today by Dr. W. W. Hagerty, Dean of the College of Engineering at the University of Texas, that the Midwestern Conference on Fluid and Solid Mechanics will be held on the Austin campus on September 9-11, 1959. This meeting, which will be the Sixth Conference on Fluid Mechanics and the Fourth Conference on Solid Mechanics, forms another element in the series held in the midwestern region of the United States. Previous conferences, at approximately two-year intervals, have been held at the Universities of Illinois, Ohio State, Minnesota, Purdue and Michigan.

Although the conferences are held on a regional basis, there are no restrictions on attendance or on the residence of authors of technical papers. A General Conference Committee for handling initial arrangements consists of Dr. Walter L. Moore, Professor of Civil Engineering; Dr. Enrico G. Volterra, Professor of Engineering Mechanics; and Dr. Milton J. Thompson, Professor of Aeronautical Engineering. Abstracts and titles of proposed papers should be submitted to Professor Thompson as chairman before March 1, 1959. Complete manuscripts will be required by May 1, 1959, so as to make possible the distribution of the Proceedings at the opening of the Conference. Inquiries concerning other details of the Conference should also be addressed to Professor Thompson.

The University of Texas, as a part of its support of the Conference, has arranged for a series of invited lectures on important aspects of mechanics. Dr. Samuel A. Schaaf of the University of California will present a lecture on "Recent Progress in Rarefied Gas Dynamics Research", while Dr. Lloyd H. Donnell of the Illinois Institute of Technology will talk on "New Developments in Shell Theory". At a dinner meeting, Dr. Sydney Goldstein of Harvard University will speak on "Recent Progress in Applied Mechanics".

Arrangements are being made for entertainment programs for conferees and their ladies. Tours of University laboratories and a visit to the Southwest Research Institute in San Antonio are also being planned.

A number of technical societies and research organizations have joined with the University of Texas in co-sponsoring the Midwestern Conference. These include the American Institute of Chemical Engineers, the American Mathematical Society, the American Meteorological Society, the American Physical Society, the American Institute of Physics, the American Society of Civil Engineers, the American Society of Mechanical Engineers, the Institute of the Aeronautical Sciences, the Society for Experimental Stress Analysis, the Society of Petroleum Engineers, the Office of Scientific Research of the Air Force, the Office of Naval Research, the Office of Ordnance Research of the Army, and the National Science Foundation.

FOURTH U. S. CONGRESS OF APPLIED MECHANICS—1962

The Fourth U. S. National Congress of Applied Mechanics will be held on the Berkeley campus of the University of California during June 18-21, 1962. The cooperating societies and other interested organizations are urged not to schedule conflicting meetings on these dates.

Research workers in the theoretical and applied mechanics of solids and fluids are invited to submit papers for consideration by the Editorial Committee. Further announcements concerning the preparation of papers and deadlines for submission will be made as the Congress draws nearer.

The members of the organizing committee on the Berkeley campus are:

Professor W. Goldsmith, Secretary
Professor E. V. Laitone, Treasurer
Professor R. M. Rosenberg, Chairman of the Editorial Committee
Professor W. W. Soroka, General Chairman

Inquiries regarding the Congress should be addressed to Professor W. Goldsmith, Secretary, Division of Mechanics and Design, University of California, Berkeley 4, California.

SOUTH AFRICAN PUBLICATIONS

The South African Institution of Civil Engineers has recently published a group of papers entitled "1957—58 Symposium on Expansive Clays".

This publication contains six papers together with discussions and authors' replies reprinted from the September, December, 1957 and the June, 1958, Transactions of the South African Institution of Civil Engineers. The publication amounts to 68 pages and costs Li-1-0.

EARTHQUAKE PUBLICATIONS

The Earthquake Engineering Research Institute announces the following new publications:

1. "Bibliography of Engineering Seismology", second edition, by Edward P. Hollis, 1958. This bibliography contains over 2,000 entries and cross-references. Most of the more important works are annotated. It is a very thorough coverage of the English language literature as well as a good representation of the Japanese, Russian, and other foreign language publications. This book is of major importance to all doing research and development work in any phase of earthquake engineering. The practicing consulting engineer will find it useful in studies of the earthquake hazard. Cloth, xii plus 144 pages, 7 inches by 10-1/2 inches, $4.50.
2. "Bibliography of Effects of Soil Conditions on Earthquake Damage", by C. Martin Duke, 1958. This is a very detailed bibliography in this important field, and over 275 specific entries are included. All items are well annotated which makes the publication quite useful when searching for specific topics. Much of the little known Japanese research is brought to light, some of it for the first time to the English reader. Paper, iii plus 47 pages, 7 inches by 10-1/2 inches, $1.00.

Both of the above publications ordered at one time—$5.00. For further information about these publications, contact:

Earthquake Engineering Research Institute
465 California Street
San Francisco 4, California

PAN-AMERICAN CONFERENCE

An announcement has been received that the First Pan-american Conference on Soil Mechanics and Foundation Engineering will be held at the University City of Mexico on September 7 to 12, 1959. The purpose of the conference is to exchange knowledge and experiences among technicians of the American Continent in this branch of Civil Engineering. Papers will be presented and discussed at the Conference on the subjects of "Foundations on clays", "Foundations on sandy soils", "Earth dams", and "Embankments and cuts for highways and airports". Soil Mechanics papers on other subjects will be accepted and will be included in the Proceedings but will not be discussed at the Conference.

The Conference is sponsored by the Mexican Society of Soil Mechanics the President of which is Dr. Leonardo Zeevaert. The Executive Committee of the Conference includes:

President:	Dr. Nabor Carrillo
General Director:	Ing. Eulalio Juarez Badillo
Technical Director:	Ing. Francisco Zamora M.
Director of Finances:	Ing. Angel Carrillo Flores
Director of Relations:	Sr. Tomas Gurza
Secretary General:	Ing. Enrique Tamez

Correspondence about the Conference should be addressed to:

Ing. Enrique Tamez, Secretario
Comité Ejecutivo del I Congreso Panamericano
de Mecánica de Suelos y Cimentaciones
Instituto de Ingeniería Ciudad Universitaria
México 20, D. F.
Mexico

JUNE NEWSLETTER

Deadline date for arrival at this office of contributions for the June Newsletter: April 20, please.

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